Surveying and Levelling

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Introduction

1.1 DEFINITIONS

Surveying is the art of determining the relative positions of different objects on the surface of the earth by measuring the horizontal distances between them, and by preparing a map to any suitable scale. Thus, in this discipline, the measurements are taken only in the horizontal plane.

Levelling is the art of determining the relative vertical distances of different points on the surface of the earth. Therefore, in levelling, the measurements are taken only in the vertical plane.

1.2 OBJECT OF SURVEYING

The aim of surveying is to prepare a map to show the relative positions of the objects on the surface of the earth. The map is drawn to some suitable scale. It shows the natural features of a country, such as towns, villages, roads, railways, rivers, etc. Maps may also include details of different engineering works, such as roads, railways, irrigation canals, etc.

1.3 USES OF SURVEYING

Surveying may be used for the following various applications.

1. To prepare a topographical map which shows the hills, valleys, rivers, villages towns, forests, etc. of a country.
2. To prepare a cadastral map showing the boundaries of fields, houses and other properties.
3. To prepare an engineering map which shows the details of engineering works such as roads, railways, reservoirs, irrigation canals, etc.
4. To prepare a military map showing the road and railway communications with different parts of a country. Such a map also shows the different strategic points important for the defence of a country.
5. To prepare a contour map to determine the capacity of a reservoir and to find the best possible route for roads, railways, etc.
6. To prepare a geological map showing areas including underground resources.
7. To prepare an archeological map including places where ancient relics exist.

1.4 CLASSIFICATION OF SURVEYING

A. Primary Classification

Surveying is primarily classified as under:

1. Plane surveying, and
2. Geodetic surveying.

1. Plane surveying We know that the shape of the earth is spheroidal. Thus, the surface is obviously curved. But in plane surveying, the curvature of the earth is not taken into consideration. This is because plane surveying is carried out over a small area. So, the surface of the earth is considered as plane. In such surveying, a line joining any two points is considered to be straight. The triangle formed by any three points is considered as a plane triangle and the angles of the triangle are assumed to be plane angles. Plane surveying is conducted by state agencies like the Irrigation Department, Railway Department, etc. Plane surveying is done on an area of less than 250 km^2.

2. Geodetic surveying In geodetic surveying, the curvature of the earth is taken into consideration. It is extended over a large area. The line joining any two points is considered as a curved line. The triangle formed by any three points is considered to be spherical and the angles of the triangle are assumed to be spherical angles. Geodetic surveying is conducted by the Survey of India department, and is carried out over an area exceeding 250 km^2.

B. Secondary Classification

1. Based on instruments:
   - (a) Chain surveying,
   - (b) Compass surveying,
   - (c) Plane table surveying,
   - (d) Theodolite surveying,
   - (e) Tacheometric surveying, and
   - (f) Photographic surveying.
2. Based on methods:
   - (a) Triangulation surveying, and
   - (b) Traverse surveying.
3. Based on object:
   - (a) Geological surveying,
   - (b) Mine surveying,
   - (c) Archaeological surveying, and
   - (d) Military surveying.
4. Based on nature of field:
   - (a) Land surveying,
   - (b) Marine surveying, and
   - (c) Astronomical surveying.

Again, land surveying is divided into the following classes:

(i) Topographical surveying, which is done to determine the natural features of a country.

(ii) Cadastral surveying, which is conducted in order to determine the boundaries of fields, estates, houses, etc.

(iii) City surveying, which is carried out to locate the premises, streets, water supply and sanitary systems, etc.

(iv) Engineering surveying, which is done to prepare detailed drawings of projects involving roads, railways, etc.

1.5 GENERAL PRINCIPLE OF SURVEYING

The general principles of surveying are:

1. To work from the whole to the part, and
2. To locate a new station by at least two measurements (linear or angular) from fixed reference points.

1. According to the first principle, the whole area is first enclosed by main stations (i.e. controlling stations) and main survey lines (i.e. controlling lines). The area is then divided into a number of parts by forming well conditioned triangles. A nearly equilateral triangle is considered to be the best well-conditioned triangle. The main survey lines are measured very accurately with a standard chain. Then the sides of the triangles are measured. The purpose of this process of working is to prevent accumulation of error. During this procedure, if there is any error in the measurement of any side of a triangle, then it will not affect the whole work. The error can always be detected and eliminated.

But, if the reverse process (i.e. from the part to the whole) is followed, then the minor errors in measurement will be magnified in the process of expansion and a stage will come when these errors will become absolutely uncontrollable.

2. According to the second principle, the new stations should always be fixed by at least two measurements (linear or angular) from fixed reference points. Linear measurements refer to horizontal distances measured by chain or tape. Angular measurements refer to the magnetic bearing or horizontal angle taken by a prismatic compass or theodolite.

In chain surveying, the positions of main stations and directions of main survey lines are fixed by tie lines and check lines.

1.6 METHODS OF LINEAR MEASUREMENT

The following methods are generally employed for linear measurements:

1. By pacing or stepping For rough and speedy work, distances are measured by pacing, i.e. by counting the number of walking steps of a man. The walking step of a man is considered as 2.5 ft or 80 cm. This method is generally employed in the reconnaissance survey of any project.

2. By passometer A small instrument just like a stop watch, the passometer is used for counting the number of steps automatically by some mechanical device. It offers an improvement over the normal pacing method when a very long distance is to be measured and when it becomes very tedious to count and extremely difficult to remember the number of steps.

3. By speedometer This is used in automobiles for recording distances.

4. By perambulator It is a wheel fitted with a fork and handle. The wheel is graduated and shows a distance per revolution. There is a dial which records the number of revolutions. Thus the distance can be ascertained.

5. By chaining This is an accurate and common method of measuring distance. In this method, the distances are directly measured in the field by chain or tape. The various types of chains and tapes are described later, as is the method of chaining.

1.7 ACCESSORIES FOR LINEAR MEASUREMENTS

1. Ranging rods Rods which are used for ranging (i.e. the process of making a line straight) a line are known as ranging rods. Such rods are made of seasoned timber or seasoned bamboo. Sometimes GI pipes of 25 mm diameter are also used as ranging rods. They are generally circular in section, of diameter 25 mm and length 2 m. Sometimes wooden ranging rods are square in section. The rod is divided into equal parts of 20 cm each and the divisions are painted black and white or red and white alternately so that the rod is visible from a long distance. The lower end of the rod is pointed or provided with an iron shoe (Fig. 1.1).

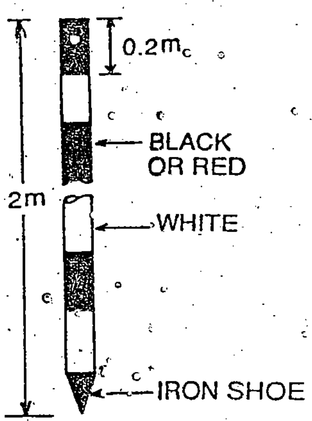


Fig. 1.1 Ranging Rod

2. Chains A chain is prepared with 100 or 150 pieces of galvanised mild steel wire of diameter 4 mm. The ends of the pieces are bent to form loops. Then the pieces are connected together with the help of three oval rings, which make the chain flexible. Two brass handles are provided at the two ends of the chain. Tallies are provided at every 10 or 25 links for facility of counting. 'One link' means the distance between the centres of adjacent middle rings (Fig. 1.2).

The following are the different types of chains:

(a) Metric chain,
(b) Steel band,

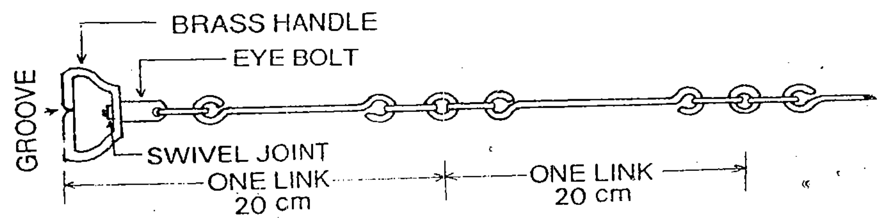


Fig. 1.2 Chain

(c) Engineers' chain,
(d) Gunter's chain, and
(e) Revenue chain.

(a) Metric Chain: Metric chains are available in lengths of 20 m and 30 m. The 20 m chain is divided into 100 links, each of 0.2 m. Tallies are provided at every 10 links (2 m). This chain is suitable for measuring distances along fairly level ground. The arrangement of tallies is shown in Fig. 1.3(a).

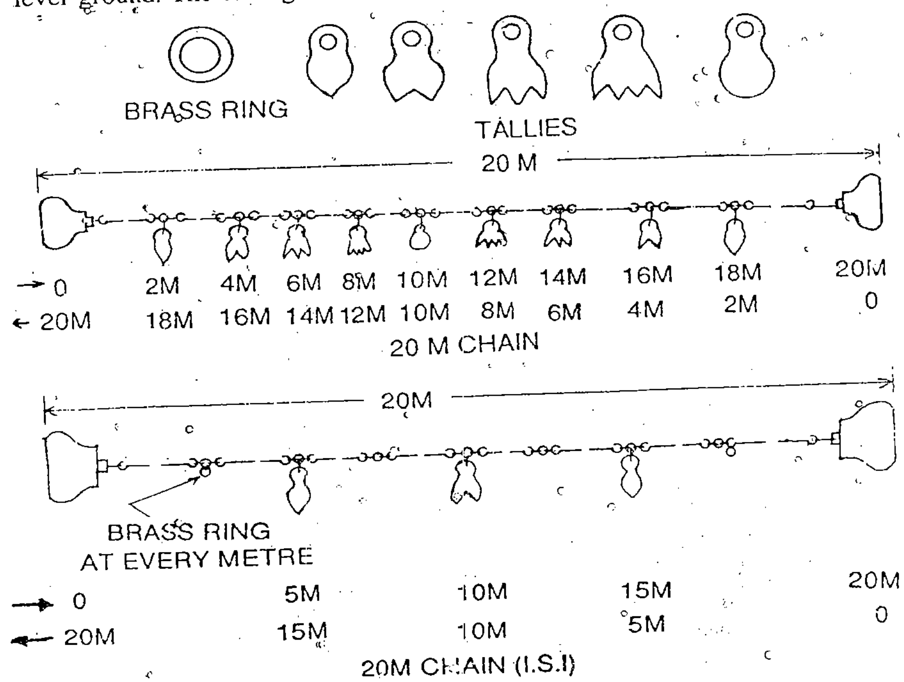


Fig. 1.3 (a) 20 m Chain (b) 20 m Chain (ISI)

You may see from the arrangement of tallies that the central tally is round and that the other tallies have one, two, three or four teeth. So, each tooth may correspond to two different readings when considered from opposite ends. Therefore, during the measurement, the surveyor should bear in mind the position of the central tally.

As per ISI recommendations, tallies should be provided after every 5 m and brass rings after every 1 m. In Fig. 1.3(b), the central tally has two teeth and the tallies on opposite sides of it have one tooth each.

The 30 m chain is divided into 150 links. So, each link is of 0.2 m. The tallies are provided after every 25 links (5 m). A round brass ring is fixed after every metre. This chain is heavy and is also suitable for measuring distances along fairly level ground. Here the central tally has three teeth (Fig. 1.4).

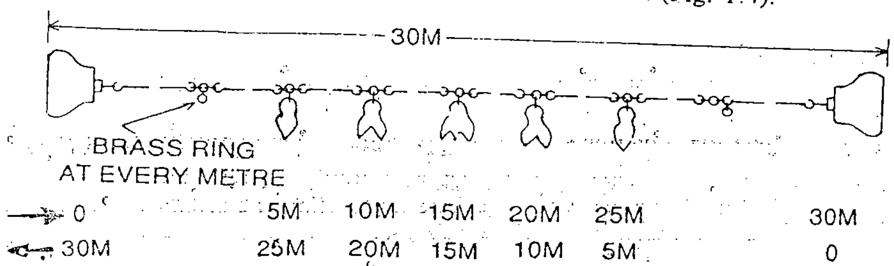


Fig. 1.4 30 m Chain

(b) Steel Band: It consists of a ribbon of steel of width 16 mm and of length 20 or 30 m. It has a brass handle at each end. It is graduated in metres, decemetres, and centimetres on one side and has 0.2 m links on the other. The steel band is used in projects where more accuracy is required.

(c) Engineers' Chain: The engineer's chain is 100 ft long and is divided into 100 links. So, each link is of 1 ft. Tallies are provided at every 10 links (10 ft), the central tally being round. Such chains were previously used for all engineering works.

(d) Gunters' Chain: It is 66 ft long and divided into 100 links. So, each link is of 0.66 ft. It was previously used for measuring distances in miles and furlongs.

(e) Revenue Chain: The revenue chain is 33 ft long and divided into 16 links. It is mainly used in cadastral survey.

Chains have the following advantages:
(a) They can be read easily and quickly.
(b) They can withstand wear and tear.
(c) They can be easily repaired or rectified in the field.

They have the following disadvantages:
(a) They are heavy and take too much time to open or fold.
(b) They become longer or shorter due to continuous use.
(c) When the measurement is taken in suspension, the chain sags excessively.

Steel bands have the following advantages:
(a) They are very light and easy to open or fold.
(b) They maintain their standard length even after continuous use.
(c) When the measurement is taken in suspension, they sag slightly.

They have the following disadvantages:
(a) If handled carelessly, they break easily.
(b) They cannot be repaired in the field.
(c) They cannot be read easily.

3. Tapes The following are the different types of tapes:
(a) Cloth or linen tape,
(b) Metallic tape,
(c) Steel tape, and
(d) Invar tape.

(a) Cloth or Linen Tape: Such a tape is made of closely woven linen and is varnished to resist moisture. It is 15 mm wide and available in lengths of 10 and 15 m. This tape is generally used for measuring offsets and for ordinary works.

(b) Metallic Tape: When linen tape is reinforced with brass or copper wires to make it durable, then it is called a metallic tape. This tape is available in lengths of 15, 20 and 30 m. It is wound on a leather case with a brass handle at the end. It is commonly used for all survey works.

(c) Steel Tape: The steel tape is made of steel ribbon of width varying from 6 to 16 mm. The commonly available lengths are 10, 15, 20, 30 and 50 m. It is graduated in metres, decimetres and centimetres. It is not used in the field, but chiefly for standardising chains and for measurements in constructional works.

(d) Invar Tape: Invar tape is made of an alloy of steel (64 %) and nickel (36%). Its thermal coefficient is very low. Therefore, it is not affected by change of temperature. It is made in the form of a ribbon of width 6 mm and is available in lengths of 30, 50 and 100 m. It is used at places where maximum precision is required. It is generally used in the triangulation survey conducted by the Survey of India department.

4. Arrows Arrows are made of tempered steel wire of diameter 4 mm. One end of the arrow is bent into a ring of diameter 50 mm and the other end is pointed. Its overall length is 400 mm. Arrows are used for counting the number of chains while measuring a chain line (Fig. 1.5).

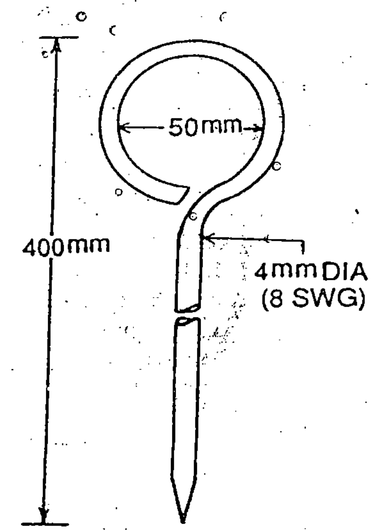


Fig. 1.5 Arrow

1.8 RANGING

The process of establishing intermediate points on a straight line between two end points is known as ranging. Ranging must be done before a survey line is chained. Ranging may be done by direct observation by the naked eye or by line ranger or by theodolite. Generally, ranging is done by the naked eye with the help of three ranging rods.

Ranging may be of two kinds:

1. Direct, and
2. Indirect or reciprocal.

1. Direct ranging When intermediate ranging rods are fixed on a straight line by direct observation from end stations, the process is known as direct ranging. Direct ranging is possible when the end stations are intervisible. The following procedure is adopted for direct ranging.

Assume that A and B are two end stations of a chain line, where two ranging rods are already fixed. Suppose it is required to fix a ranging rod at the intermediate point P on the chain line in such a way that the points A, P and B are in the same straight line. The surveyor stands about 2 m behind the ranging rod at A by looking towards the line AB. The assistant holds a ranging rod at P vertically at arm's length. The rod should be held lightly by the thumb and forefinger. Now, the surveyor directs the assistant to move the ranging rod to the left or right until the three ranging rods come exactly in the same straight line. To check the non-verticality of the rods, the surveyor bends down and looks through the bottom of the rods. The ranging will be perfect, when the three ranging rods coincide and appear as a single rod. When the surveyor is satisfied that the ranging is perfect, he signals the assistant to fix the ranging rod on the ground by waving both his hands up and down. Following the same procedure, the other ranging rods may be fixed on the line (Fig. 1.6).

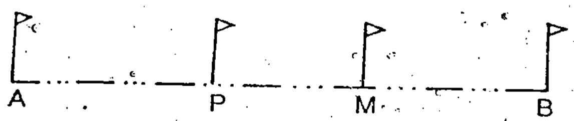


Fig. 1.6 Direct Ranging

2. Indirect or reciprocal ranging When the end stations are not intervisible due to there being high ground between them, intermediate ranging rods are fixed on the line in an indirect way. This method is known as indirect ranging or reciprocal ranging. The following procedure is adopted for indirect ranging.

Suppose A and B are two end stations which are not intervisible due to high ground existing between them. Suppose it is required to fix intermediate points between A and B. Two chain men take up positions at R_1 and S_1 with ranging rods in their hands. The chainman at R_1 stands with his face towards B so that he can see the ranging rods at S_1 and B. Again, the chainman at S_1 stands with his face towards A so that he can see the ranging rods at R_1 and A. Then the chainmen proceed to range the line by directing each other alternately. The chainman at R_1 directs the chainman at S_1 to come to the position S_2 so that R_1, S_2 and B are in the same straight line. Again, the chainman at S_2 directs the chainman at R_1 to move to the position at R_2 so that S_2, R_2 and A are in the same straight line. By directing each other alternately in this manner, they change their positions every time until they finally come to the positions R and S, which are in the straight line AB. This means the points A, R, S and B are in the same straight line (Fig. 1.7).

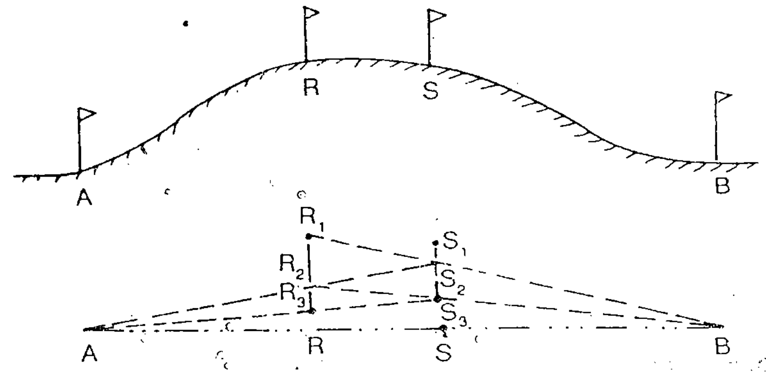


Fig. 1.7 Reciprocal Ranging

1.9 UNFOLDING AND FOLDING A CHAIN

1. Unfolding To open a chain, the strap is unfastened and the two brass handles are held in the left hand and the bunch is thrown forward with the right hand. Then one chainman stands at the starting station by holding one handle and another moves forward by holding the other handle until the chain is completely extended.

2. Folding To fold the chain, a chainman should move forward by pulling the chain at the middle. Then the two halves of the chain will come side by side. After this, commencing from the central position of the chain, two pairs of links are taken at a time with the right hand and placed on the left hand alternately in both directions. Finally, the two brass handles will appear at the top. The bunch should be then fastened by the strap.

1.10 TESTING A CHAIN

Due to continuous use, a chain may be elongated or shortened. So, the chain should be tested and adjusted accordingly. If full adjustment is not possible, then the amount of shortening (known as 'too short') and elongation (known as 'too long') should be noted clearly for necessary correction applicable to the chain.

For testing the chain, a test gauge is established on a level platform with the help of a standard steel tape. The steel tape is standardised at 20°C and under a tension of 8 kg. The test gauge consists of two pegs having nails at the top and fixed on a level platform a required distance apart (say 20 or 30 m). The incorrect chain is fully stretched by pulling it under normal tension (say about 8 kg) along the test gauge. If the length of the chain does not tally with standard length, then an attempt should be made to rectify the error. Finally, the amount of elongation or shortening should be noted (Fig. 1.8).

The allowable error is about 2 mm per 1 m length of the chain. The overall length of the chain should be within the following permissible limits:

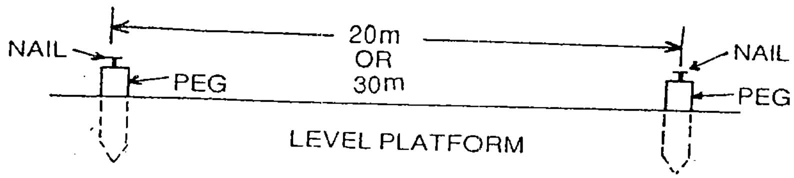


Fig. 1.8 Test Gauge

20 m chain: ± 5 mm 30 m chain: ± 8 mm

1.11 ADJUSTMENT OF CHAIN

Chains are adjusted in the following ways:

1. When the chain is too long, it is adjusted by:
   (a) Closing up the joints of the rings,
   (b) Hammering the elongated rings,
   (c) Replacing some old rings by new rings, and
   (d) Removing some of the rings.
2. When the chain is too short, it is adjusted by:
   (a) Straightening the bent links,
   (b) Opening the joints of the rings,
   (c) Replacing the old rings by some larger rings, and
   (d) Inserting new rings where necessary.

1.12 DEGREE OF ACCURACY IN CHAINING

The degree of accuracy in chaining is expressed as a ratio called the chaining ratio. The chaining ratio may be 1/1000, 1/2000, etc.

For example, if there is an error of 0.25 m during the measurement of a total length of 500 m,

$$\text{Chaining ratio} = \frac{0.25}{500} = \frac{25}{500 \times 100} = \frac{1}{2,000}$$

Some permissible limits of error:

1. For measurement with steel band— $\frac{1}{2,000}$
2. For measurement with tested chain— $\frac{1}{1,000}$
3. In normal conditions— $\frac{1}{500}$
4. For rough work— $\frac{1}{250}$

1.13 LEADER AND FOLLOWER

The chainman at the forward end of the chain, who drags the chain forward, is known as the leader. The duties of the leader are as follows.

1. To drag the chain forward with some arrows and a ranging rod,
2. To fix arrows on the ground at the end of every chain, and
3. To obey the instructions of the follower.

The chainman at the rear end of the chain, who holds the zero end of the chain at the station, is known as the follower. The duties of the follower are:

1. To direct the leader at the time of ranging,
2. To carry the rear handle of the chain, and
3. To pick up the arrows inserted by the leader.

1.14 METHOD OF CHAINING ON LEVEL GROUND

Before starting the chaining operation two ranging rods should be fixed on the chain line, at the end stations. The other ranging rod, should be fixed near the end of each chain length, during the ranging operation.

To chain the line, the leader moves forward by dragging the chain and by taking with him a ranging rod and ten arrows. The follower stands at the starting station by holding the other end of the chain. When the chain is fully extended, the leader holds the ranging rod vertically at arm's length. The follower directs the leader to move his rod to the left or right until the ranging rod is exactly in line. Then the follower holds the zero end of the chain by touching the station peg. The leader stretches the chain by moving it up and down with both hands, and finally places it on the line. He then inserts an arrow on the ground at the end of the chain and marks with a cross ('×').

Again, the leader moves forward by dragging the chain with nine arrows and the ranging rod. At the end of the chain, he fixes another arrow as before. As the leader moves further, the follower picks up the arrows which were inserted by the leader. During chaining, the surveyor or an assistant should conduct the ranging operation.

In this way, chaining is continued. When all the arrows have been inserted and the leader has none left with him, the follower hands them over to the leader; this should be noted by the surveyor. To measure the remaining fractional length, the leader should drag the chain beyond the station and the follower should hold the zero end of the chain at the last arrow. Then the odd links should be counted.

1.15 METHOD OF CHAINING ON SLOPING GROUND

Horizontal distances are required in surveying. So, in chaining along a sloping ground, the horizontal distances between two stations are measured carefully by applying some convenient methods. The following are the different methods that are generally employed:

A. Direct method or stepping method, and
B. Indirect method.

A. Direct Method

This method is applied when the slope of the ground is very steep. In this method, the sloping ground is divided into a number of horizontal and vertical strips, like steps. So, this method is also known as the stepping method. The lengths of horizontal portions are measured and added to get the total horizontal distance between the points. The steps may not be uniform, and would depend on the nature of the ground.

Procedure Suppose the horizontal distance between points A and B in Fig. 1.9 is to be measured. The line AB is first ranged properly. Then, the follower holds the zero end of the tape at A. The leader selects a suitable length AP_1 so that P_1

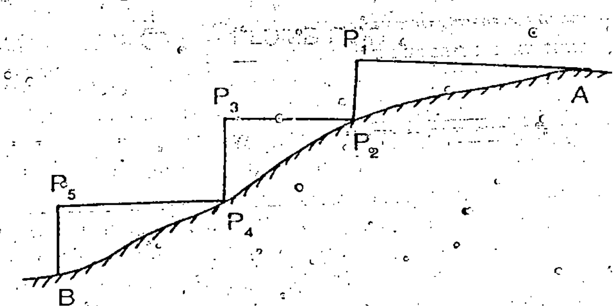


Fig. 1.9 Stepping Method

is at chest height and AP_1 is just horizontal. The horizontality is maintained by eye estimation, by tri-square or by wooden set-square. The point P_2 is marked on the ground by plumb-bob so that P_1 is just over P_2. The horizontal length AP_1 is noted. Then the follower moves to the position P_2 and holds the zero end of the tape at that point. Again the leader selects a suitable length P_2P_3 in such a way that P_2P_3 is horizontal and P_3P_4 vertical. Then the horizontal lengths P_2P_3 and P_4P_5 are measured.

So, the total horizontal length, $AB = AP_1 + P_2P_3 + P_4P_5$

B. Indirect Method

When the slope of the ground surface is long and gentle, the stepping method is not suitable. In such a case, the horizontal distance may be obtained by the following processes:

1. By measuring the slope with a clinometer,
2. By applying hypotenusal allowance, and
3. By knowing the difference of level between the points.

1. Measuring the slope with a clinometer A clinometer is a graduated semicircular protractor. It consists of two pins (P_1 and P_2 in Fig. 1.10) for sighting the object.

A plumb bob is suspended from point O with a thread. When the straight edge is just horizontal, the thread passes through 0°. When the straight edge is tilted, the thread remains vertical, but passes through a graduation on the arc which shows the angle of slope.

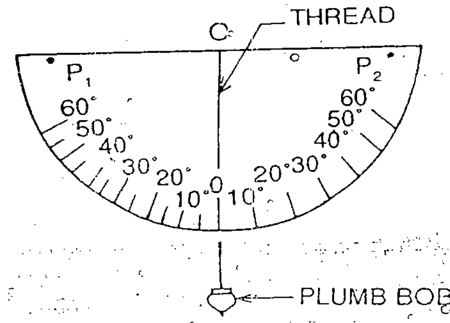


Fig. 1.10 Clinometer

Measurement of slope and sloping distance: Suppose C and D are two points on sloping ground. Two ranging rods are fixed at these points. Then two other points C_1 and D_1 are marked on the ranging rods so that $CC_1 = DD_1$.

The clinometer is placed in such a way that its centre just touches the mark C_1. The clinometer is then inclined gradually until the points P_1, P_2 and D_1 are in the same straight line. At this position the thread of the clinometer will show an angle which is the angle of slope of the ground. Suppose this angle is α. The sloping distance CD is also measured. Let it be l (Fig. 1.11).

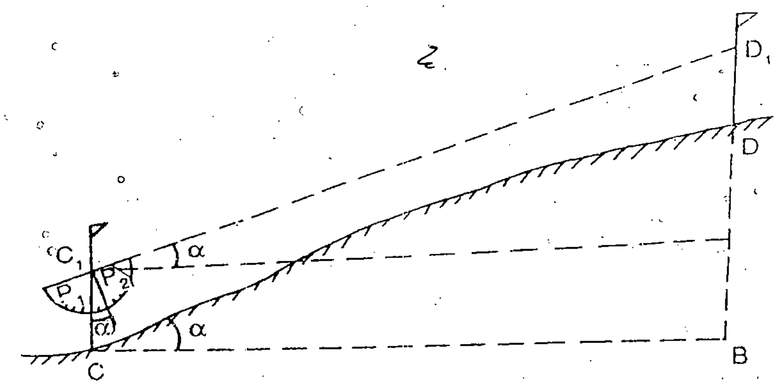


Fig. 1.11 Slope Measurement

The required horizontal distance, $CB = l \cos \alpha$

2. Applying hypotenusal allowance In this method, the slope of the ground is first found out by using the clinometer or Abney level. Hypotenusal allowance is then made for each tape length (Fig. 1.12).

Let θ = angle of slope measured by clinometer or Abney level

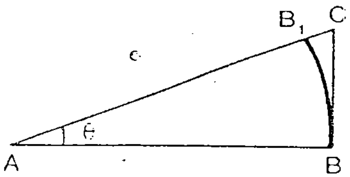


Fig. 1.12

$$AB = AB_1 = 20 \text{ m} = 100 \text{ links}$$

$$AC = AB \sec\theta = 100 \sec\theta$$

$$B_1C = AC - AB_1$$

$$= 100 \sec\theta - 100$$

$$= 100 (\sec\theta - 1)$$

The amount $100 (\sec\theta - 1)$ is said to be the hypotenusal allowance.

While chaining along the slope, one chain would be actually located at B_1. But the arrow should be placed at C after making hypotenusal allowance. The next chain length will start from C. The same principle is followed until the end of the line is reached.

3. Knowing the difference of level Suppose, A, B, C and D are different points on sloping ground. The difference of level between these points is determined by a levelling instrument. Let the respective differences be h_1, h_2 and h_3. Then the sloping distances AB, BC and CD are measured. Let the distances l_1, l_2 and l_3 respectively (Fig. 1.13).

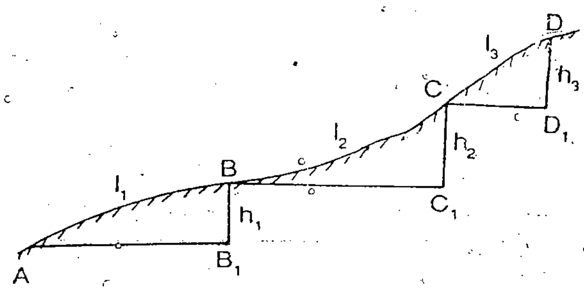


Fig. 1.13

The required horizontal distances are given by

$$AB_1 = \sqrt{l_1^2 - h_1^2} \qquad BC_1 = \sqrt{l_2^2 - h_2^2}$$

$$CD_1 = \sqrt{l_3^2 - h_3^2}$$

Total horizontal distance = $AB_1 + BC_1 + CD_1$.

1.16 OBSTACLE IN CHAINING

A chain line may be interrupted in the following situations:

1. When chaining is free, but vision is obstructed,
2. When chaining is obstructed, but vision is free, and
3. When chaining and vision are both obstructed.

1. Chaining free but vision obstructed Such a problem arises when a rising ground or a jungle area interrupts the chain line. Here the end stations are not intervisible. There may be two cases.

Case I: The end stations may be visible from some intermediate points on the rising ground. In this case, reciprocal ranging is resorted to, and the chaining is done by the stepping method. (These are discussed in Secs 1.8 and 1.15.)

Case II: The end stations are not visible from intermediate points when a jungle area comes across the chain line. In this case the obstacle may be crossed over using a random line as explained below:

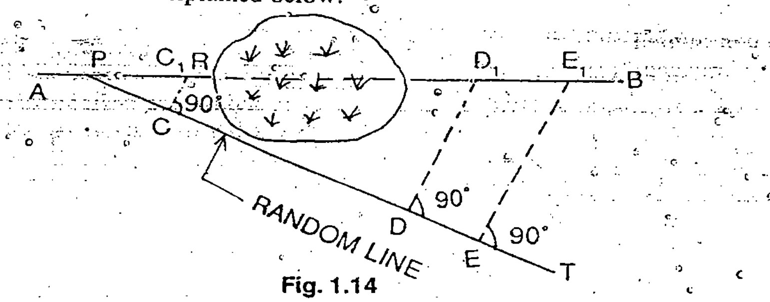


Fig. 1.14

Let AB be the actual chain line which cannot be ranged and extended because of interruption by a jungle. Let the chain line be extended up to R. A point P is selected on the chain line and a random line PT is taken in a suitable direction. Points C, D and E are selected on the random line, and perpendiculars are projected from them. The perpendicular at C meets the chain line at C_1.

Theoretically, the perpendiculars at D and E will meet the chain line at D_1 and E_1. Now, the distances PC, PD, PE and CC_1 are measured (Fig. 1.14).

From triangles PDD_1 and PCC_1,

$$\frac{DD_1}{PD} = \frac{CC_1}{PC}$$

$$DD_1 = \frac{CC_1}{PC} \times PD \qquad (1)$$

Again, from triangles PEE_1 and PCC_1

$$\frac{EE_1}{PE} = \frac{CC_1}{PC}$$

$$EE_1 = \frac{CC_1}{PC} \times PE \qquad (2)$$

From (1) and (2), the lengths DD_1 and EE_1 are calculated. These calculated distances are measured along the perpendiculars at D and E. Points D_1 and E_1 should lie in the chain line AB, which can be extended accordingly.

$$\text{Distance } PE_1 = \sqrt{PE^2 + EE_1^2}$$

2. Chaining obstructed but vision free Such a problem arises when a pond or a river comes across the chain line. The situations may be tackled in the following ways.

Case I: When a pond interrupts the chain line, it is possible to go around the obstruction.

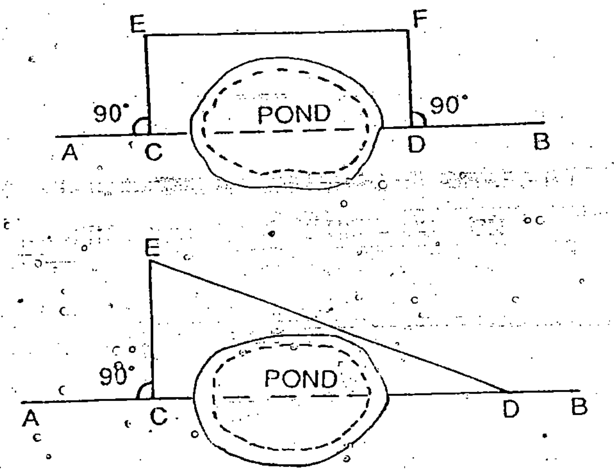


Fig. 1.15 (a), (b)

Suppose AB is the chain line. Two points C and D are selected on it on opposite banks of the pond. Equal perpendiculars CE and DF are erected at C and D. The distance EF is measured.

Here, $CD = EF$ (Fig. 1.15(a))

The pond may also be crossed by forming a triangle as shown in Fig. 1.15(b). A point C is selected on the chain line. The perpendicular CE is set out at C, and a line ED is suitably taken. The distances CE and ED are measured.

so $$CD = \sqrt{ED^2 - CE^2}$$

Case II: Sometimes it is not possible to go around the obstruction.

(a) Imagine a small river comes across the chain line. Suppose AB is the chain line. Two points C and D are selected on this line on opposite banks of the river. At C a perpendicular CE is erected and bisected at F. A perpendicular is set out at E and a point G is so selected on it that D, F and G are in the same straight line.

From triangles DCF and GEF,

$$GE = CD$$

This distance GE is measured, and thus the distance CD is obtained indirectly (Fig. 1.16 (a)).

(b) Consider the case when a large river interrupts the chain line.

Let AB be the chain line. Points C, D and E are selected on this line such that D and E are on opposite banks of the river. The perpendiculars DF and CG are erected on the chain line in such a way that E, F and G are on the same straight line. The line FH is taken parallel to CD.

Now, from triangles DEF and HFG,

$$\frac{ED}{DF} = \frac{FH}{HG} \qquad \text{where, } FH = CD$$

$$ED = \frac{FH}{HG} \times DF \qquad CH = DF$$

$$= \frac{CD}{CG - DF} \times DF \qquad HG = CG - CH$$

$$\therefore \quad HG = CG - DF$$

The distances CD, DF and CG are measured. Thus the required distance ED can be calculated (Fig. 1.16(b)).

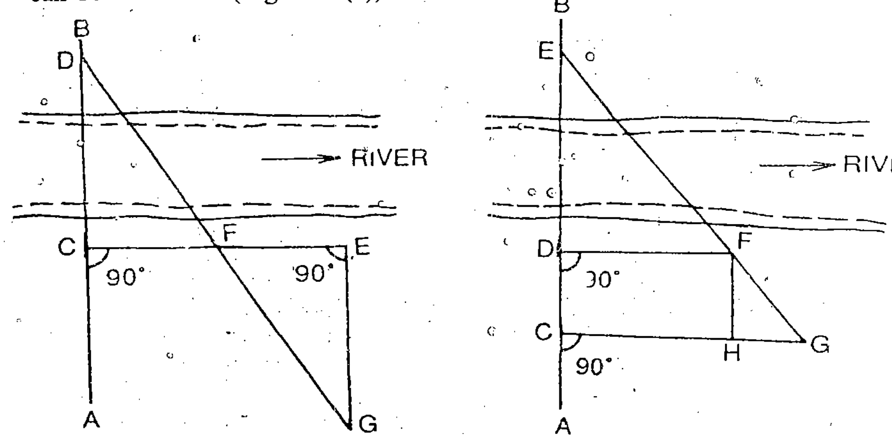


Fig. 1.16 (a) and (b)

3. Chaining and vision both obstructed Such a problem arises when a building comes across the chain line. It is solved in the following manner.

Suppose AB is the chain line. Two points C and D are selected on it at one side of the building. Equal perpendiculars CC_1 and DD_1 are erected. The line C_1D_1 is extended until the building is crossed. On the extended line, two points E_1 and F_1 are selected. Then perpendiculars E_1E and F_1F are so erected that

$$E_1E = F_1F = D_1D = C_1C$$

Thus, the points C, D, E and F will lie on the same straight line AB.

Here, $DE = D_1E_1$

The distance D_1E_1 is measured, and is equal to the required distance DE (Fig. 1.17).

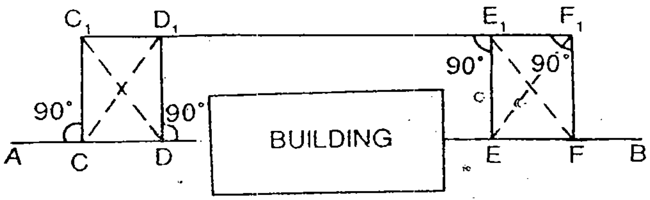


Fig. 1.17

1.17 CONCEPTION OF MAGNETIC BEARING

When a magnetic needle is suspended freely, it will show a direction which is known as the magnetic meridian.

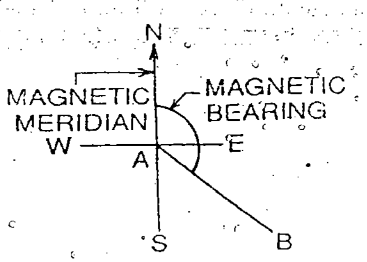


Fig. 1.18

The angle of a line makes with the magnetic maridian, it is known as the magnetic bearing of the line (Fig. 1.18).

Magnetic bearing is expressed as: (a) whole circle bearing, and (b) reduced or quadrantal bearing.

1. Whole circle bearing (WCB) In this system, the magnetic bearing of a line is measured clockwise from the north point up to the line.

For example, in Fig. 1.19,

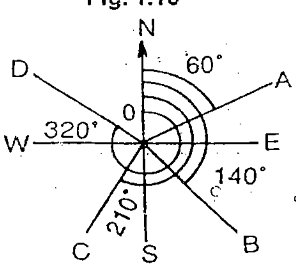


Fig. 1.19

WCB of OA = 60°; WCB of OB = 140°
WCB of OC = 210°; WCB of OD = 320°

2. Reduced bearing (RB) In this system the bearing is measured clockwise or counterclockwise from the north or south towards the east or west. Here four quadrants are considered and are denoted as NE, NW, SE and SW. The values of RB may lie between 0° and 90°, but the quadrants must be mentioned too (Fig. 1.20).

For example, in Fig. 1.20,

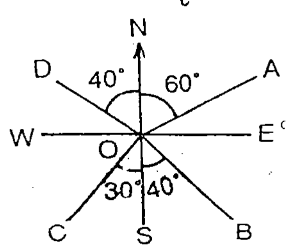


Fig. 1.20

RB of OA = N 60° E
RB of OB = S 40° E
RB of OC = S 30° W
RB of OD = N 40° W

Note: Bearings will be studied in detail in the chapter on "compass surveying". A basic idea has been given here only to help you understand how problems caused by obstacles in chaining may be solved.

1.18 TO FIND THE HEIGHT OF AN OBJECT BY USING ONLY TAPE AND RANGING RODS

We shall now discuss a simple method of finding the height of an object, when it is accessible and there is level ground in front of it.

Let PT be the tower whose height is to be determined. Let A and B be two ranging rods fixed on level ground some distance apart. Let T be the base and P the peak of the tower. Two marks C and D are made on the ranging rods at A and B, so that AC = BD. Then, looking through the line CD, a mark E is made on the tower. The peak of the tower P is sighted from C, so that C, D_1 and P are in the same straight line. The point D_1 is marked on the ranging rod at B. Now the distances AB, AT and CD_1 are measured. From triangles PEC and D_1DC (Fig. 1.21),

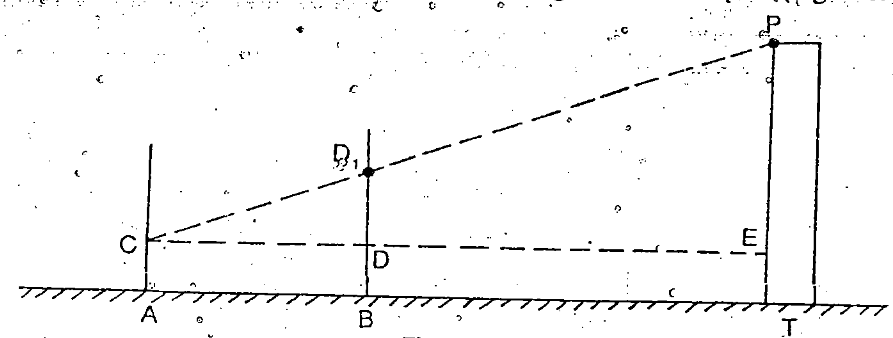


Fig. 1.21

$$\frac{PE}{CE} = \frac{DD_1}{CD} \qquad \text{Here } CD = AB$$

$$CE = AT$$

or $$PE = \frac{DD_1}{CD} \times CE$$

i.e. $$PE = \frac{DD_1}{AB} \times AT$$

Thus PE is calculated. Then ET is measured using the tape.

So, Height of tower = PE + ET

1.19 ERRORS AND MISTAKES IN CHAINING

Errors in chaining may be caused due to variation in temperature and pull, defects in instruments, etc. They may be either:

1. compensating, or
2. cumulative.

1. Compensating errors Errors which may occur in both directions (i.e. both positive and negative) and which finally tend to compensate are known as compensating errors. These errors do not affect survey work seriously. They are proportional to $\sqrt{L}$, where L is the length of the line. Such errors may be caused by:

(a) Incorrect holding of the chain,
(b) Horizontality and verticality of steps not being properly maintained during the stepping operation;
(c) Fractional parts of the chain or tape not being uniform throughout its length, and
(d) Inaccurate measurement of right angles with chain and tape.

2. Cumulative errors Errors which may occur in the same direction and which finally tend to accumulate are said to be cumulative. They seriously affect the accuracy of the work, and are proportional to the length of the line (L). The errors may be positive or negative.

Positive errors: When the measured length is more than the actual length, (i.e. when the chain is too short), the error is said to be positive. Such errors occur due to:

(a) The length of chain or tape being shorter than the standard length,
(b) Slope correction not being applied,
(c) Correction for sag not being made,
(d) Measurement being taken with faulty alignment, and
(e) Measurement being taken in high winds with the tape in suspension.

Negative errors: When the measured length of the line is less than the actual length (i.e. when the chain is too long), the error is said to be negative. These errors occur when the length of the chain or tape is greater than the standard length due to the following reasons.

(a) The opening of ring joints,
(b) The applied pull being much greater than the standard pull,
(c) The temperature during measurement being much higher than the standard temperature,
(d) Wearing of connecting rings, and
(e) Elongation of the links due to heavy pull.

3. Mistakes Errors occurring due to the carelessness of the chainman are called 'mistakes'. The following are a few common mistakes.

(a) Displacement of arrows: Once an arrow is withdrawn from the ground during chaining, it may not be replaced in proper position, if required due to some reason.
(b) A full chain length may be omitted or added. This happens when arrows are lost or wrongly counted.
(c) A reading may be taken from the wrong end of the chain. This happens when the tooth of the tally is noted without observing the central tally (i.e. when the tooth is noted from the wrong end).
(d) The numbers may be read from the wrong direction; for instance, a '6' may be read as a '9'.
(e) Some numbers may be called wrongly. For example, 50.2 may be called as "fifty-two" without the decimal point being mentioned.
(f) While making entries in the field book, the figures may be interchanged due to carelessness: for instance, 245 may be entered instead of 254.

1.20 PRECAUTIONS AGAINST ERRORS AND MISTAKES

The following precautions should be taken to guard against errors and mistakes.

1. The point where the arrow is fixed on the ground should be marked with a cross (×).
2. The zero end of the chain or tape should be properly held.
3. During chaining, the number of arrows carried by the follower and leader should always tally with the total numbers of arrows taken.
4. While noting the measurement from the chain, the teeth of the tally should be verified with respect to the correct end.
5. The chainman should call the measurement loudly and distinctly and the surveyor should repeat them while booking.
6. Measurements should not be taken with the tape in suspension in high winds.
7. In stepping operations, horizontality and verticality should be properly maintained.
8. Ranging should be done accurately.
9. No measurement should be taken with the chain in suspension.
10. Care should be taken so that the chain is properly extended.

1.21 CHAIN AND TAPE CORRECTIONS

A. Tape Correction

1. Temperature correction (C_t) This correction is necessary because the length of the tape or chain may be increased or decreased due to rise or fall of temperature during measurement. The correction is given by the expression

$$C_t = \alpha (T_m - T_0) L$$

where,

C_t = correction for temperature, in metres
α = coefficient of thermal expansion
T_m = temperature during measurement in degrees centigrade or celsius
T_0 = temperature at which the tape was standardised, in degrees centigrade or celsius
L = length of tape, in metres

The sign of correction may be positive or negative according as T_m is greater or less than T_0.

When α for the steel tape is not given, it may be assumed to be 11×10^{-6} per degree centigrade or celsius.

2. Pull correction (C_p) During measurement, the applied pull may be either more or less than the pull at which the chain or tape was standardised. Due to the elastic property of materials, the strain will vary according to the variation of applied pull, and hence necessary correction should be applied. This correction is given by the expression

$$C_p = \frac{(P_m - P_0)\,L}{A \times E}$$

where C_p = pull correction in metres
P_m = pull applied during measurement, in kilograms
P_0 = pull at which the tape was standardised, in kilograms
L = length of tape, in metre
A = cross-sectional area of tape, in square centimetres
E = modulus of elasticity (Youngs' modulus)

The sign of correction will be positive or negative according as P_m is greater or less than P_0.

When E is not given, it may be assumed 2.1×10^6 kg/cm^2.

3. Slope correction (C_h) Slope correction is calculated as follows.

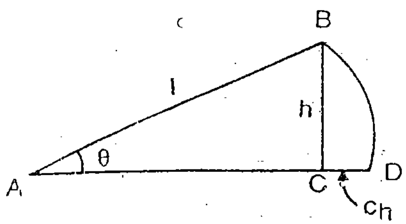


Fig. 1.22

$$C_h = l - \sqrt{l^2 - h^2} \quad \text{(exact)} \quad (1)$$

$$= l\,(1 - \cos\theta) \quad \text{(exact)} \quad (2)$$

$$= \frac{h^2}{2l} \quad \text{(approx)} \quad (3)$$

This correction is always negative.

4. Sag correction (C_s) This correction is necessary when the measurement is taken with the tape in suspension (i.e. in the form of a catenary). It is given by the expression

$$C_s = \frac{L(\omega L)^2}{24 n^2 P_m^2} \quad (1)$$

when unit weight is given

and

$$C_s = \frac{L W^2}{24 n^2 P_m^2} \quad (2)$$

when total weight is given

where, C_s = sag correction, in metres

L = length of tape or chain, in metres
ω = weight of tape per unit length, in kilograms per metre
W = total weight of tape, in kilograms
n = number of spans
P_m = pull applied during measurement, in kilograms

The sign of correction is always negative.

5. Normal tension (P_n) The tension at which the effect of pull is neutralised by the effect of sag is known as normal tension. At this tension, the elongation due to pull is balanced by the shortening due to sag. So, equating the expressions for correction for pull and sag, we have

$$\frac{(P_n - P_0)\,L}{AE} = \frac{L(\omega L)^2}{24 P_m^2} \quad \text{(considering } n = 1\text{)}$$

where, P_n = normal pull or tension

Here the value of P_n may be determined by trial, by forming an equation by putting the known values.

$$\frac{(P_n - P_0)\,L}{AE} = \frac{L(\omega L)^2}{24 P_n^2} \quad \text{(considering } n = 1\text{)}$$

or

$$\frac{(P_n - P_0)}{AE} = \frac{W^2}{24 P_n^2}$$

or

$$(P_n - P_0)\,P_n^2 = \frac{W^2 AE}{24}$$

By substituting the values of P_0, W, A and E, an equation will be obtained in the following form:

$$x P_n^3 \pm y P_n^2 \pm C = 0$$

Then, the value of P_n is to be determined by satisfying the equation by trial and error.

B. Chain Correction

1. Correction applied to incorrect length It is given by the expression

$$\text{True length of line (TL)} = \left(\frac{L'}{L}\right) \times \text{measured length (ML)}$$

where L = Standard or true length of chain
L' = True length ± error
$= L \pm e$ (e = error in chain or tape, i.e. when it is too long or too short)

Use the positive sign when the chain or tape is too long, the negative sign when it is too short.

2. Correction of incorrect area The correction to be applied in this case is given by the expression

$$\text{True area} = \left(\frac{L'}{L}\right)^2 \times \text{measured area}$$

3. Hypotenusal allowance This is explained in Sec. 1.15.
Hypotenusal allowance per tape = L (sec θ − 1)

where L = length of tape
θ = slope of the ground

This allowance is always added to the tape length.

1.22 WORKED OUT PROBLEMS ON CHAIN AND TAPE CORRECTIONS

Problem 1 The distance between two points, measured with a 20 m chain, was recorded as 327 m. It was afterwards found that the chain was 3 cm too long. What was the true distance between the points?

Solution Given data:

True length of chain, L = 20 m
Error in chain, e = 3 cm = 0.03 m, too long
$L' = L + e = 20 + 0.03 = 20.03$ m
Measured length = 327 m

$$\text{True length of line} = \frac{L'}{L} \times \text{ML}$$

$$= \frac{20.03}{20} \times 327 = 327.49 \text{ m}$$

Problem 2 The distance between two stations was 1,200 m when measured with a 20 m chain. The same distance when measured with 30 m chain was found to be 1,195 m. If the 20 m chain was 0.05 m too long, what was the error in the 30 m chain?

Solution Let us consider the 20 m chain.

L = 20 m L' = 20 + 0.05 = 20.05 m
Measured length = 1,200 m

$$\text{True length of line} = \frac{20.05}{20} \times 1{,}200 = 1{,}203 \text{ m}$$

Let us now consider the 30 m chain.

L = 30 m L' = ?

True length of line 1,203 m (as obtained from 20 m chain)
Measured length = 1,195 m.
From the relation

$$\text{TL} = \frac{L'}{L} \times \text{ML}$$

$$1{,}203 = \frac{L'}{30} \times 1{,}195$$

$$L' = \frac{1{,}203 \times 30}{1{,}195} = 30.20 \text{ m}$$

Now, L' is greater than L. So, the chain is too long.
Amount of error, e = 30.20 − 30 = + 0.20 m

Problem 3 A line was measured by a 20 m chain which was accurate before starting the day's work. After chaining 900 m, the chain was found to be 6 cm too long. After chaining a total distance of 1,575 m, the chain was found to be 14 cm too long. Find the true distance of the line.

Solution First part:

$$L = 20 \text{ m}$$

$$L' = 20 + \frac{0 + 0.06}{2} \text{ (considering mean elongation)}$$

$$= 20.03 \text{ m}$$

$$\text{ML} = 900 \text{ m}$$

$$\text{TL} = ?$$

$$\text{TL} = \frac{L'}{L} \times \text{ML}$$

$$= \frac{20.03}{20} \times 900 = 901.35 \text{ m}$$

Second part:

$$L = 20 \text{ m}$$

$$L' = 20 + \frac{0.06 + 0.14}{2} = 20.1 \text{ m}$$

$$\text{ML} = 1{,}575 - 900 = 675 \text{ m}$$

$$\text{TL} = \frac{20.1}{20} \times 675 = 678.375 \text{ m}$$

True distance = 901.350 + 678.375 = 1,579.725 m

Problem 4 On a map drawn to a scale of 50 m to 1 cm, a surveyor measured the distance between two stations as 3,500 m. But it was found that by mistake he had used a scale of 100 m to 1 cm. Find the true distance between the stations.

Solution First method:

As the surveyor used the scale of 100 m to 1 cm,

$$\text{Distance between stations on map} = \frac{3500}{100} = 35 \text{ cm}$$

As the actual scale of map is 50 m to 1 cm,

True distance on the ground = 35 × 50 = 1,750 m

Second method:

$$\text{True distance} = \frac{\text{RF of wrong scale}}{\text{RF of correct scale}} \times \text{measured length}$$

$$\text{True distance} = \frac{\frac{1}{100 \times 100}}{\frac{1}{50 \times 100}} \times 3{,}500$$

$$= \frac{50 \times 100}{100 \times 100} \times 3{,}500$$

∴ True distance = 50 × 35 = 1,750 m

Problem 5 An old map was plotted to a scale of 40 m to 1 cm. Over the years, this map has been shrinking, and a line originally 20 cm long is only 19.5 cm long at present. Again the 20 m chain was 5 cm too long. If the present area of the map measured by planimeter is 125.50 cm^2, find the true area of the land surveyed.

Solution According to the given conditions,

19.5 cm on the map was originally 20 cm.

Therefore, 1 cm on the map was originally $= \frac{20}{19.5}$ cm, and

$$1 \text{ cm}^2 \text{ on the map was originally} = \frac{(20)^2}{(19.5)^2} \text{ cm}^2$$

$$125.50 \text{ cm}^2 \text{ was originally} = \frac{(20)^2}{(19.5)^2} \times 125.50 = 132.0184 \text{ cm}^2$$

Scale of map was 1 cm = 40 m

⇒ 1 cm^2 = 1,600 m^2

Area on the ground = 1,600 × 132.0184

= 211,229.44 m^2

Since the chain was 0.05 m too long,

$$\text{True area} = \frac{(20.05)^2}{(20)^2} \times 211{,}229.44 = 212{,}286.90 \text{ m}^2$$

= 21.2286 hectares

(1 hectare = 10,000 m^2)

Problem 6 A steel tape was exactly 30 m long at 20°C when supported throughout its length under a pull of 10 kg. A line was measured with this tape under a pull of 15 kg and at a mean temperature of 32°C and found to be 780 m long. The cross-sectional area of the tape = 0.03 cm^2, and its total weight = 0.693 kg. α for steel = 11 × 10^{-6} per °C and E for steel = 2.1 × 10^6 kg/cm^2. Compute the true length of the line if the tape was supported during measurement (i) at every 30 m (ii) at every 15 m. (WBSC 1989)

Solution Given data:

L	= 30 m	A	= 0.03 cm^2
T_0	= 20°C	α	= 11 × 10^{-6} per °C
P_0	= 10 kg	E	= 2.1 × 10^6 kg/cm^2
P_m	= 15 kg	W	= 0.693 kg
T_m	= 32°C	ML	= 780 m

(a) When supported at every 30 m:

Total correction per tape length is to be found out first. Here, n = 1.

(i) Temperature correction, $C_t = \alpha(T_m - T_0)\, L$

$= 11 \times 10^{-6}\ (32 - 20) \times 30$

$= 0.00396$ m (+ve)

(ii) Pull correction, $C_p = \frac{(P_m - P_0)L}{A \times E}$

$= \frac{(15 - 10) \times 30}{0.03 \times 2.1 \times 10^6} = 0.00238$ m (+ve)

(iii) Sag correction, $C_s = \frac{LW^2}{24 n^2 P_m^2}$

$= \frac{30 \times (0.693)^2}{24 \times (15)^2} = 0.00267$ m (−ve)

Total correction = + 0.00396 + 0.00238 − 0.00267

= + 0.00367 m (too long)

so $L' = L + e = 30.00367$ m

True length $= \frac{L'}{L} \times$ ML

$= \frac{30.00367}{30} \times 780 = 780.094$ m

(b) When supported at every 15 m:

Here, span n = 2

Let us find out the correction per tape length.

(i) Temperature correction = 0.00396 m (+ve) as before

(ii) Pull correction = 0.00238 m (+ve) as before

(iii) Sag correction $= \frac{LW^2}{24 n^2 P_m^2}$

$$= \frac{30 \times (0.693)^2}{24 \times 2^2 \times (15)^2} = 0.00067 \text{ m } (-\text{ve})$$

Total correction = + 0.00396 + 0.00238 − 0.00067

= + 0.00567 m (too long)

so $L' = L + e = 30.00567$

$$\text{True length} = \frac{30.00567}{30} \times 780 = 780.147 \text{ m}$$

Problem 7 A 20-m steel tape was standardised on flat ground, at a temperature of 20°C and under a pull of 15 kg. The tape was used in catenary at a temperature of 30°C and under a pull of P kg. The cross-sectional area of the tape is 0.22 cm², and its total weight is 400 g. The Young's modulus and coefficient of linear expansion of steel are 2.1×10^6 kg/cm² and 11×10^{-6} per °C respectively. Find the correct horizontal distance if P is equal to 10 kg. (WBSC 1988)

Solution Given data:

$L = 20$ m	$A = 0.02$ cm²
$T_0 = 20$°C	$\alpha = 11 \times 10^{-6}$ per °C
$P_0 = 15$ kg	$E = 2.1 \times 10^{-6}$ kg/cm²
$T_m = 30$°C	$W = 400$ g = 0.4 kg
$P = 10$ kg	$n = 1$

Here, applied pull $P = 10$ kg.

(a) Temperature correction, $C_t = \alpha(T_m - T_0) L$

$= 11 \times 10^{-6} (30 - 20) 20$

$= 11 \times 10^{-6} \times 10 \times 20$

$= 0.00220$ m (+ve)

(b) Pull correction, $C_p = \dfrac{(P - P_0) L}{A \times E}$

$$= \frac{(10 - 15) 20}{0.02 \times 2.1 \times 10^6}$$

$$= -\frac{5 \times 20}{0.02 \times 2.1 \times 10^6}$$

$= -0.00238$ m (−ve)

(c) Sag correction, $C_s = \dfrac{LW^2}{24n^2P^2}$ $(n = 1)$

$$= \frac{20 \times (0.4)^2}{24 \times (10)^2} = 0.00133 \text{ m } (-\text{ve})$$

Total correction = + 0.00220 − 0.00238 − 0.00133 = − 0.00151 m

Correct horizontal distance = 20 − 0.00151 = 19.99849 m

Problem 8 A 30 m steel tape was standardised at a temperature of 20°C and under a pull 5 kg. The tape was used in catenary at a temperature of 25°C and under a pull of P kg. The cross-sectional area of the tape is 0.02 cm², its weight per unit length is 22 g/m, Young's modulus = 2×10^{-6} kg/cm², $\alpha = 11 \times 10^{-6}$ per °C. Find the correct horizontal distance, if P is equal to (i) 5 kg, and (ii) 11 kg. (WBSC 1986)

Solution Given data:

$L = 30$ m	$A = 0.02$ cm²
$T_0 = 20$°C	$E = 2 \times 10^6$ kg/cm²
$P_0 = 5$ kg	$\alpha = 11 \times 10^{-6}$ per °C
$T_m = 25$°C	$W = 22$ g/m
P = (i) 5 kg (ii) 11 kg	Total weight $W = 22 \times 30$ = 660 g = 0.66 kg
$n = 1$	

(a) When applied pull $P = 5$ kg:

(i) Temperature correction $C_t = \alpha(T_m - T_0) L$

$= 11 \times 10^{-6} (25 - 20) 30$

$= 0.00165$ m (+ve)

(ii) Pull correction $= \dfrac{(P - P_0) L}{AE}$

$$= \frac{(5 - 5) \times 30}{0.02 \times 2 \times 10^6} = 0$$

(iii) Sag correction, $C_s = \dfrac{LW^2}{24n^2p^2} = \dfrac{30 \times (0.66)^2}{24 \times (5)^2}$ $(n = 1)$

$= + 0.02178$ m (−ve)

Total correction = + 0.00165 − 0.02178 = − 0.02013 m

Correct horizontal distance = 30 − 0.02013 = 29.97987 m

(b) When applied pull $P = 11$ kg:

(i) Temperature correction $C_t = 0.00165$ m (+ve) as before.

(ii) Pull correction, $C_p = \dfrac{(P - P_0) L}{AE}$

$$= \frac{(11 - 5) \times 30}{0.02 \times 2 \times 10^6} = 0.0045 \text{ m } (+\text{ ve})$$

(iii) Sag correction, $C_s = \dfrac{LW^2}{24n^2p^2}$ $(n = 1)$

$$= \frac{30 \times (0.66)^2}{24 \times (11)^2}$$

$= 0.00449$ m (−ve)

$$\text{Total correction} = +0.00165 + 0.00450 - 0.00449 = +0.00166 \text{ m}$$

$$\text{Correct horizontal distance} = 30 + 0.00166 = 30.00166 \text{ m}$$

Problem 9 A steel tape was exactly 20 m long at 20°C when supported throughout its length under a pull of 5 kg. A line measured with this tape under a pull of 16 kg and at a mean temperature of 32°C, was found to be 680 m long. Assuming the tape is supported at every 20 m, find the true length of the line. Given that: (i) Cross-sectional area of tape = 0.03 cm², (ii) $E = 2.1 \times 10^6$ kg/cm², (iii) $\alpha = 11 \times 10^{-6}$ per °C, and (iv) weight of tape = 10 g/cc. (WBSC 1982)

Solution Given data:

$L = 20$ m	$A = 0.03$ cm²
$T_0 = 20°C$	$\alpha = 11 \times 10^{-6}$ per °C
$P_0 = 5$ kg	$E = 2.1 \times 10^6$ cm²
$T_m = 32°C$	Given weight = 10 g/cc
$P_m = 16$ kg	Total $W = 0.03 \times 20 \times 100 \times 10$
ML = 680 m	= 600 g = 0.6 kg
$n = 1$	

Correction per tape length:

(a) Temperature correction, $C_t = \alpha (T_m - T_0) L$

$$= 11 \times 10^{-6} (32 - 20) \times 20$$

$$= 0.00264 \text{ m} \quad (+\text{ve})$$

(b) Pull correction, $C_p = \dfrac{(P_m - P_0) L}{AE}$

$$= \frac{(16 - 5) \times 20}{0.03 \times 2.1 \times 10^6} = 0.00349 \text{ m} \quad (+\text{ve})$$

(c) Sag correction $C_s = \dfrac{L(W)^2}{24 P_m^2}$ $(n = 1)$

$$= \frac{20 \times (0.6)^2}{24 \times (16)^2} = 0.00117 \text{ m} \quad (-\text{ve})$$

$$\text{Total correction} = +0.00264 + 0.00349 - 0.00117 = +0.00496 \text{ m}$$

Actual length of tape, $L' = 20.00496$ m

True length of line $= \dfrac{L'}{L} \times \text{ML}$

$$= \frac{20.00496}{20} \times 680 = 680.169 \text{ m}$$

Problem 10 A 30 m steel tape was standardised at a temperature of 20°C and under a pull of 10 kg. The tape was used in catenary to fix a distance of 28 m between two points at 40°C and under a pull of 5 kg. Given that the cross-sectional area of the tape = 0.02 cm², total weight 470 g, Young's modulus of steel = 2.1×10^6 kg/cm², and coefficient of linear expansion = 11×10^{-6} per °C, (a) find the correct distance between the points, and (b) find the value of pull for which the measured distance would be equal to the correct distance. (WBSC 1984)

Solution Given data:

Distance between two points = 28 m

So, here

$L = 28$ m (span length)	
$T_0 = 20°C$	$A = 0.02$ cm²
$P_0 = 10$ kg	$E = 2.1 \times 10^6$ kg/cm²
$T_m = 40°C$	$\alpha = 11 \times 10^{-6}$ per °C
$P_m = 5$ kg	Total weight = 470 g
	Weight for 28 m = $\dfrac{470 \times 28}{30}$
	$W = 439$ g = 0.439 kg

(a) We have to first find the correct distance.

(i) Temperature correction $= 11 \times 10^{-6} (40 - 20) \times 28$

$$= 0.00616 \text{ m } (+\text{ve})$$

(ii) Pull correction $= \dfrac{(5 - 10)\, 28}{0.02 \times 2.1 \times 10^6} = -0.00333$ m (–ve)

(iii) Sag correction $= \dfrac{28\, (0.439)^2}{24 \times (5)^2} = 0.00899$ m (–ve)

$$\text{Total correction} = +0.00616 - 0.00333 - 0.00899 = -0.00616 \text{ m}$$

$$\text{Correct distance} = 28 - 0.00616 = 27.99384 \text{ m}$$

(b) Now we have to find the normal tension at which the measured distance would be equal to the correct distance. This condition will be satisfied when

Pull correction = sag correction

Let, Normal tension = P_n

Then,

$$\frac{(P_n - P_0) L}{AE} = \frac{LW^2}{24 P_n^2}$$

or

$$\frac{(P_n - 10)}{0.02 \times 2.1 \times 10^6} = \frac{(0.439)^2}{24 P_n^2}$$

or

$$24 P_n^3 - 240 P_n^2 - 8{,}095 = 0$$

or

$$P_n^3 - 10 P_n^2 - 337.3 = 0$$

Now, the value of P_n has to be found out by trial.

Method of Trials: Putting the assumed values of P_n in the equation,

$$P_n^3 - 10P_n^2 - 337.3 = 0$$

We get the following results.

When $P_n = 10$,
$1000 - 1000 - 337.3 = -337.3$ which is not acceptable.

When $P_n = 11$,
$1331 - 1210 - 337.3 = -216.3$ which is not acceptable.

When $P_n = 12$,
$1728 - 1440 - 337.3 = -49.3$ which is not acceptable

When $P_n = 12.25$,
$1838.27 - 1500.63 - 337.3 = +0.34$ which may be accepted, as the equation is nearly satisfied.

So, the required pull, at which the measured distance will be equal to correct distance, is 12.25 kg.

1.23 PROBLEMS ON OBSTACLES IN CHAINING

Problem 1 A survey line CD intersects a building. To overcome the obstacle a perpendicular DE, 87 m long, is set out at D. From E, two lines EF and EG are set out at angles 50° and 65° respectively with ED. Find the lengths EF and EG such that points F and G fall on the prolongation of CD. Also find the obstructed distance DF. (WBSC 1989)

Solution

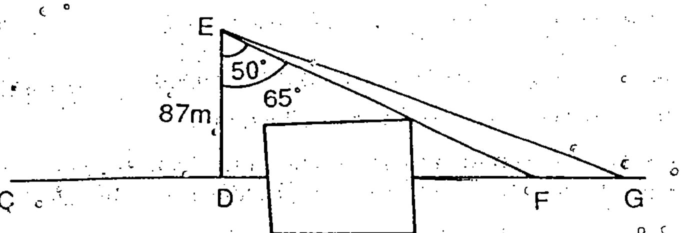


Fig. P.1.1

From Δ DEF,

$$\frac{DE}{EF} = \cos 50^\circ$$

$$EF = \frac{DE}{\cos 50^\circ} = \frac{87}{0.6428} = 135.345 \text{ m}$$

and

$$\frac{DF}{DE} = \tan 50^\circ$$

$$DF = DE \tan 50^\circ = 87 \times 1.1918 = 103.68 \text{ m}$$

From Δ DEG,

$$\frac{DE}{EG} = \cos 65^\circ$$

$$EG = \frac{DE}{\cos 65^\circ} = \frac{87}{0.4226} = 205.9 \text{ m}$$

Problem 2 P and Q are two points 367 m apart on the same bank of a river. The bearings of a tree on the other bank observed from P and Q are N 36°25′ E and N 40°35′ W, respectively. Find the width of the river if bearings of PQ are S 86°35′ E. (WBSC 1988)

Solution

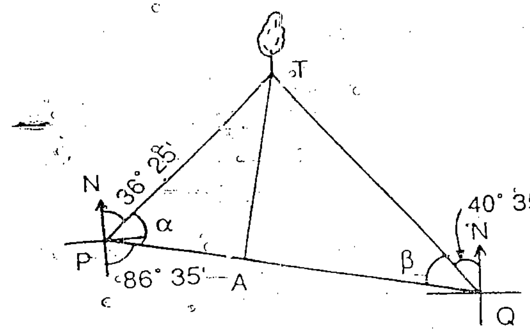


Fig. P. 1.2

Let the points P and Q be on the near side and the tree T on the far bank of the river. From T, draw a perpendicular TA to PQ. Then TA is the width of the river.

Let $PA = x$

Then, $AQ = 367 - x$

$$\alpha = 180^\circ - (36^\circ25' + 86^\circ35') = 57^\circ0'$$

$$\beta = 86^\circ35' - 40^\circ35' = 46^\circ0'$$

From Δ PTA,

$$\frac{TA}{PA} = \tan \alpha$$

$$TA = x \tan 57^\circ 0' \quad (1)$$

From Δ QTA,

$$\frac{TA}{AQ} = \tan \beta$$

$$TA = (367 - x) \tan 46^\circ0' \quad (2)$$

From (1) and (2),

$$x \tan 57^\circ 0' = (367 - x) \tan 46^\circ0'$$

or $$x \times 1.5399 = (367 - x) \times 1.0355$$

or $$2.5754\, x = 380.0285$$

$$x = 147.56 \text{ m}$$

From (1),

$$TA = 147.56 \times 1.5399 = 227.229 \text{ m}$$

So, the width of the river is 227.229 m.

Problem 3 P and Q are two points 517 m apart on the same bank of a river. The bearings of a tree on the other bank observed from P and Q are N 33°40′ E and N43°20′ W respectively. Find the width of the river if the bearings of PQ are N78° E. (WBSC 1986)

Solution

Let the points P and Q be on the near bank and the tree T on the far bank of the river. From T, draw a perpendicular drawn to PQ.

Let $PA = x$

Then, $AQ = (517 - x)$

Here $\alpha = 78°0' - 33°40' = 44°20'$

$\beta = 180° - (43°20' + 78°0')$

$= 58°40'$

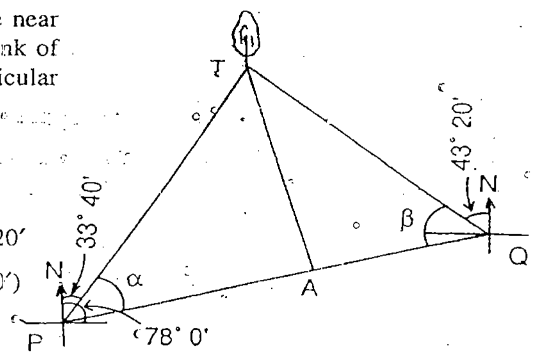


Fig. P.1.3

From triangle PTA, $\dfrac{TA}{PA} = \tan \alpha$

$$TA = x \tan 44°20' \quad (1)$$

From triangle QTA, $\dfrac{TA}{QA} = \tan \beta$

$$TA = (517 - x) \tan 58°40' \quad (2)$$

From (1) and (2), $x \tan 44°20' = (517 - x) \tan 58°40''$

or $x \times 0.9770 = (517 - x) \times 1.6426$

or $2.6196\, x = 849.224$

or $x = 324.18$ m

From (1), $TA = 324.18 \times 0.9770 = 316.724$ m

So, the width of the river is 316.724 m.

Problem 4 A survey line BAC crosses a river, A and C being on the near and opposite banks respectively. A perpendicular AD, 40 m long, is set out at A. If the bearings of AD and DC are 48°30′ and 288°30′ respectively, draw the sketch and find the bearing of the chain line BAC and also the chainage of C when that of A is 207.8 m.

Solution

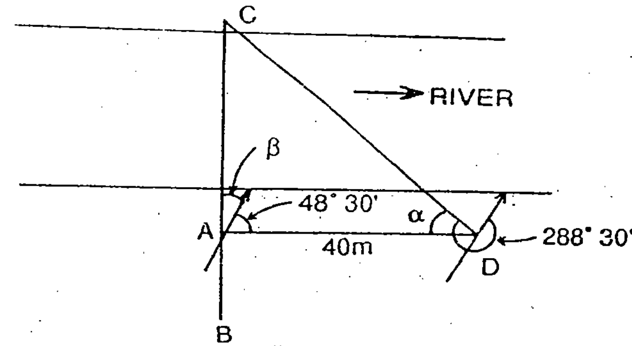


Fig. P.1.4

$$\angle ADC = \alpha = \text{FB of DC} - \text{BB of AD}$$

$$= 288°30' - (48°30' + 180°0') = 60°0'$$

$$\beta = 90°0' - 48°30' = 41°30'$$

Bearing of the chain line BAC = 360°0′ − 41°30′ = 318°30′

From triangle ADC, $\dfrac{AC}{AD} = \tan \alpha$

or $AC = 40 \times \tan 60° = 69.284$ m

Chainage of C = 207.8 + 69.284 = 277.08 m

Problem 5 A chain line ABC crosses a river, B and C being on the near and distant banks respectively. The line BM of length 75 m is set out at right angles to the chain line at B. If the bearings of BM and MC are 287°15′and 62°15′ respectively, find the width of the river.

Solution

$$\angle BMC = \text{BB of BM} - \text{FB of MC}$$

i.e. $\alpha = (287°15' - 180°0') - 62°15' = 45°0'$

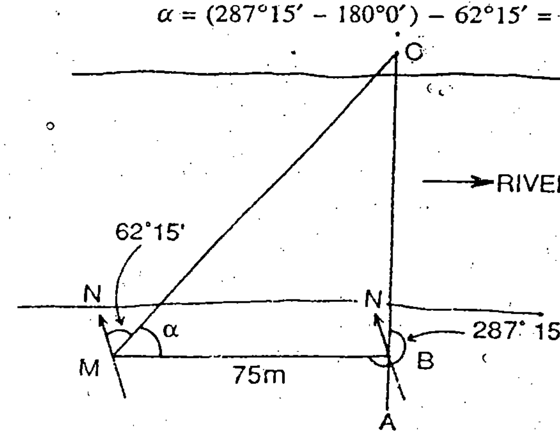


Fig. P.1.5

From triangle MBC, $\frac{BC}{BM} = \tan 45°$

$$BC = BM \tan 45° = 75 \text{ m}$$

So, the width of the river is 75 m.

Problem 6 A chain line PQ intersects a pond. Two points A and B are taken on the chain line on opposite sides of the pond. A line AC, 250 m long, is set out on the left of AB and another line AD, 300 m long, is set out on the right of AB. Points C, B and D are in the same straight line. CB and BD are 100 and 150 m long respectively. Calculate the length of AB.

Solution

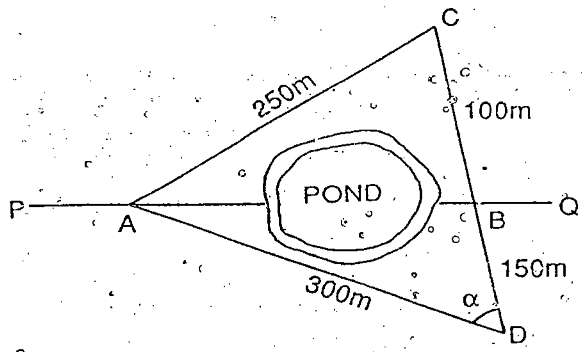


Fig. P.1.6

$$AC = 250 \text{ m}, AD = 300 \text{ m}, CD = 100 + 150 = 250 \text{ m}$$

In triangle ADC, let $\angle ADC = \alpha$

We know that $AC^2 = AD^2 + DC^2 - 2AD \times DC \cos \alpha$

$$\cos \alpha = \frac{AD^2 + DC^2 - AC^2}{2AD \times DC}$$

$$\therefore \quad \cos \alpha = \frac{300^2 + 250^2 - 250^2}{2 \times 300 \times 250} = 0.6$$

Again in triangle ADB, $AB^2 = AD^2 + DB^2 - 2AD \times DB \cos \alpha$

$$AB = \sqrt{300^2 + 150^2 - 2 \times 300 \times 150 \times 0.6}$$

$$= \sqrt{112{,}500 - 54{,}000} = 241.87 \text{ m}$$

Problem 7 **A chain line ABC crosses a river, B and C being on the near and distant banks respectively. A perpendicular BE, 50 m long, is set out at B on left of the chain line. AB is 25 m long. The bearings of C and A taken from E are 67°30′ and 157°30′ respectively. Find the chainage of C, if the chainage of B is 275.5 m.**

Solution

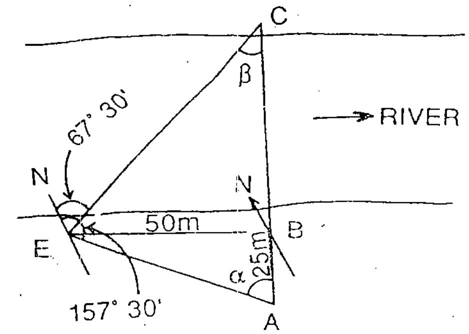


Fig. P.1.7

Here, BE = 50 m and AB = 25 m

$$\angle AEC = 157°30' - 67°30' = 90°$$

From triangle ABE, $\tan \alpha = \frac{BE}{AB} = \frac{50}{25} = 2$

$\therefore \quad \alpha = 63°26'$

and $\quad \beta = 90°0' - 63°26' = 26°34' \quad$ (as $\angle AEC = 90°$)

From triangle BEC, $\frac{BE}{BC} = \tan \beta$

$$BC = \frac{BE}{\tan \beta} = \frac{50}{\tan 26°34'} = 100 \text{ m}$$

So, chainage of C = 275.5 + 100 = 375.5 m

1.24 PROBLEMS RELATED TO SLOPING GROUND

Problem 1 The following slope distances were measured along a chain line with a 20 m steel tape;

Slope distance (m) = 17.5, 19.3, 17.8, 13.6, and 12.9

Difference of elevation between ends (m) = 2.35, 4.20, 2.95, 1.65, and 3.25

It was noted afterwards that the tape was 2.5 cm too short. Find the true horizontal distance.

Solution

$AB = \sqrt{17.5^2 - 2.35^2} = 17.34$ m $\quad B_1C = \sqrt{19.3^2 - 4.2^2} = 18.84$ m

$C_1D = \sqrt{17.8^2 - 2.95^2} = 17.56$ m $\quad D_1E = \sqrt{13.6^2 - 1.65^2} = 13.49$ m

$E_1F = \sqrt{12.9^2 - 3.25^2} = 12.48$ m

Total horizontal distance = $AB + B_1C + C_1D + D_1E + E_1F$

$= 79.71$ m

Here the steel tape was 2.5 cm too short.

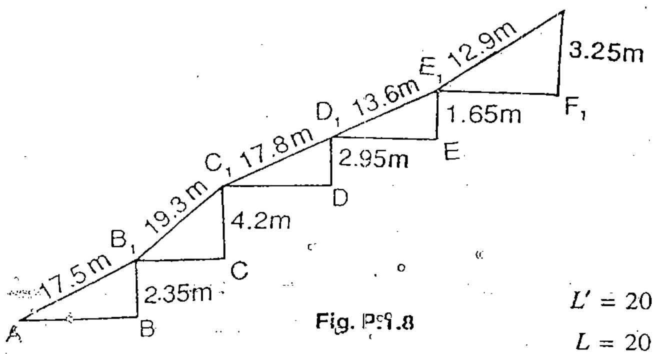


Fig. P.1.8

$L' = 20 - 0.025 = 19.975$ m

$L = 20$ m $ML = 79.71$ m

True length $= \frac{19.975}{20} \times 79.71 = 79.61$ m

Problem 2 The length of a line measured on a slope of 15° was recorded as 550 m. But it was found that the 20 m chain was 0.05 m too long. Calculate the true horizontal distance of the line.

Solution

Horizontal distance $AB = AB_1 \cos 15°$

$= 550 \times 0.9659$

$= 531.25$ m

Again $L = 20$ m

$L' = (20 + 0.05)$ m $= 20.05$ m

$ML = 531.25$ m

True length $= \frac{20.05}{20} \times 531.25$

$= 532.6$ m

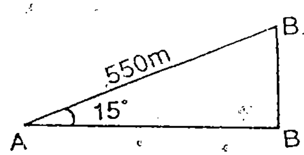


Fig. P.1.9

Problem 3 The distance between two points A and B measured along a slope was 280 m. Determine the horizontal distance between A and B when (a) the angle of slope is 10° (b) the slope is 1 in 10, and (C) the difference of level between A and B is 8 m.

Solution

(a)

Horizontal distance,

$AB = 280 \cos 10° = 275.74$ m

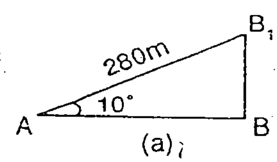


Fig. P.1.10 (a)

(b)

Horizontal distance, $AB = 280 \cos \alpha$

$= 280 \times \frac{10}{\sqrt{101}} = 278.6$ m

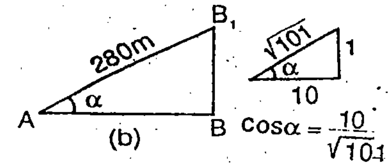


Fig. P.1.10 (b)

(c)

Horizontal distance,

$AB = \sqrt{280^2 - 8^2} = 279.9$ m

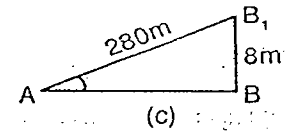


Fig. P.1.10 (c)

Problem 4 The following slope distances were measured along a chain line with a 30 m chain.

Slope distance	*Angle of slope*
28.7 m	5°
23.4 m	7°
20.9 m	10°
29.6 m	12°

It was noted afterwards that the chain was 0.025 m too short. Find the true horizontal distance.

Solution

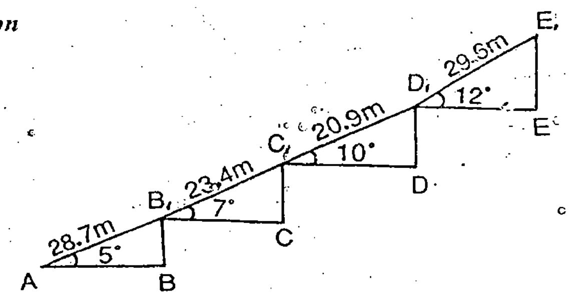


Fig. P.1.11

$AB = 28.7 \cos 5° = 28.59$ m

$B_1C = 23.4 \cos 7° = 23.22$ m

$C_1D = 20.9 \cos 10° = 20.58$ m

$D_1E = 29.6 \cos 12° = 28.95$ m

Total $= 101.34$

Total horizontal distance = 101.34 m

Again, $L = 30$ m

$L^1 = 30 - 0.025 = 29.975$ m

ML = 101.34 m

$$\text{True horizontal distance} = \frac{29.975}{30} \times 101.34 = 101.25 \text{ m}$$

1.25 TO VERIFY WHETHER A TRIANGLE IS WELL-CONDITIONED

Problem 1 The sides of a triangle are 12.0, 16.5 and 23.0 m respectively. Examine whether the triangle is well-conditioned.

Solution

Let θ_1 = acute angle opposite to smallest side

θ_2 = obtuse angle opposite to greatest side

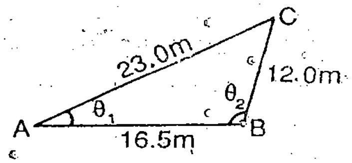


Fig. P.1.12

Now, $$\cos\theta_1 = \frac{23^2 + 16.5^2 - 12^2}{2 \times 23 \times 16.5}$$

$$= \frac{657.25}{759} = 0.866$$

or $\cos\theta_1 = \cos 30°$

$\theta_1 = 30°$

$$\cos\theta_2 = \frac{16.5^2 + 12^2 - 23^2}{2 \times 16.5 \times 12} = -\frac{112.75}{396} = -0.2847$$

or $\cos\theta_2 = -\cos 73°27'$

$= \cos(180° - 73°27') = \cos 106°33'$

∴ $\theta_2 = 106°33'$

As the obtuse angle is less than 120°, it is a well-conditioned triangle.

Problem 2 The sides of a triangle are 156, 103 and 257 m. Examine whether the triangle is well-conditioned.

Solution

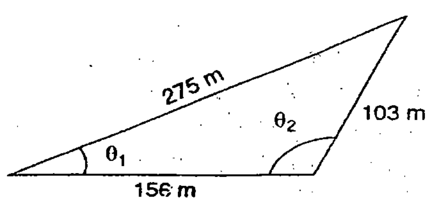


Fig. P. 1.13

Let θ_1 = acute angle opposite to smallest side

θ_2 = obtuse angle opposite to greatest side

Now, $$\cos\theta_1 = \frac{257^2 + 156^2 - 103^2}{2 \times 257 \times 156} = \frac{79{,}776}{80{,}184} = 0.9949$$

or $\cos\theta_1 = \cos 5°48'$

$\theta_1 = 5°48'$

$$\cos\theta_2 = \frac{156^2 + 103^2 - 257^2}{2 \times 156 \times 103} = -0.9679$$

or $\cos\theta_2 = -\cos 14°33'$

$= \cos(180° - 14°33') = \cos 165°27'$

∴ $\theta_2 = 165°27'$

The acute angle is less than 30° and the obtuse angle greater than 120°. So, the given triangle is not a well-conditioned.

1.26 PROBLEMS ON SCALES

Scale It is not always possible to represent the actual length of an object on a drawing. So, it is required to reduce the object, in order to accommodate it on the drawing, in some proportion. The ratio by which the actual length of the object is reduced or increased is known as the 'scale'.

Full-Size Scale If the actual length of the object is shown on the drawing, the scale used is said to be a full-size scale.

Reducing Scale If the actual length of the object is reduced in order to accommodate it on the drawing sheet the scale used is said to be a reducing scale.

Increasing or Enlarging Scale If the actual length of an object is enlarged so as to bring out its details more clearly on the drawing, the scale used is said to be an enlarging scale.

Representative fraction (RF) The ratio of the distance on the drawing to the corresponding actual length of the object is known as the representative fraction, i.e.

$$\text{RF} = \frac{\text{distance on drawing of object}}{\text{corresponding actual distance of object}}$$

(both distances in same units) i.e. in cm

For example,

If a scale is 1 cm = 10 m, then

$$\text{RF} = \frac{1}{10 \times 100} = \frac{1}{1{,}000}$$

Types of Scale Scales can be of the following four types:

(a) Plain,
(b) Diagonal,
(c) Comparative, and
(d) Vernier.

The surveyor should have a good knowledge of the plain and diagonal scales. (They are explained here.)

Plain scale This scale used to represent two successive units, such as 'kilometres, hectometres', 'metres, decimetres', 'metres, 1/10th of metre', and so on.

Diagonal scale This is a scale used to represent three successive units or one unit and its fraction up to the second place of decimals, such as 'kilometres, hectometres, decametres,' 'metres, decimetres, centimetres,' and 'metres, 1/100th of a metre,' and so on.

Length of scale The length of scale should be calculated according to the maximum length to be shown. But when the maximum length is not given, the length of scale is to be assumed within 10 cm to 15 cm in such a way that the maximum distance to be shown is divisible by 10 or 100.

Problem 1 To construct a scale to show metres and decimetres when 1 m is represented by 2.5 cm, the scale should be long enough to measure 6 m.

Solution

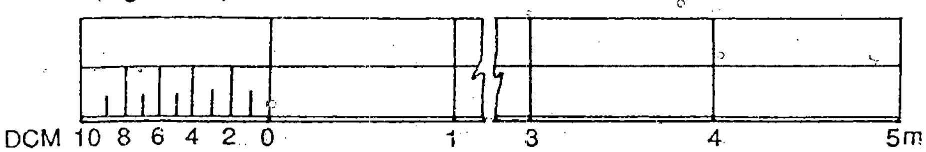
$$\text{RF} = \frac{2.5}{1 \times 100} = \frac{1}{40}$$

Max. distance to be shown = 6 m

Length of scale = RF × max. distance

$$= \frac{1}{40} \times 6 \times 100 = 15 \text{ cm}$$

Here, a length of 15 cm is divided into six parts, each representing 1 m. The division on the extreme left is divided into 10 further parts, each representing 1 dcm (Fig. P.1.14).



R F = $\frac{1}{40}$

Fig. P.1.14

Problem 2 A plan represents an area of 93,750 m² and measures 6.00 cm × 6.25 cm. Find the scale of the plot and indicate through a sketch how a suitable scale can be constructed to read up to 1 m in the plan. (WBSC 1989)

Solution 6.00 × 6.25 cm² represents 93,750 m².

∴ 1 cm² represents $\frac{93{,}750}{6.00 \times 6.25} = 2{,}500 \text{ m}^2$

or $1 \text{ cm} = \sqrt{2500}$

i.e. $1 \text{ cm} = 50 \text{ m}$

$$\text{RF} = \frac{1}{5{,}000}$$

Here, the maximum length to be shown is not given.

So, let us assume the length of the scale to be 14 cm, i.e.

$$14 \text{ cm} = 14 \times 50 = 700 \text{ m}$$

14 cm are divided into 7 divisions. Each division represents 100 m the left-hand division is further divided into 10 divisions horizontally at the base and 10 divisions vertically and then joined diagonally to get a minimum reading of 1 m.

So, the scale constructed is diagonal (Fig. P.1.15).

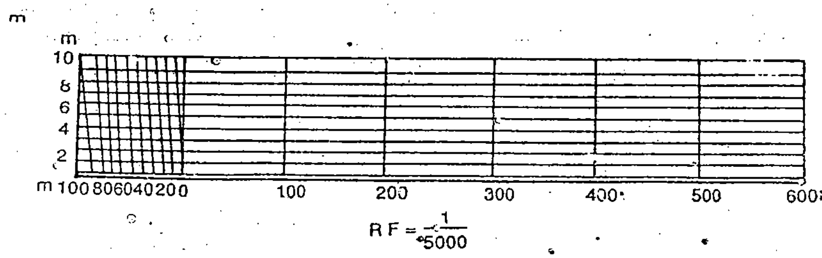


Fig. P.1.15

Problem 3 A 1.2 km long road is indicated in a map by a length of 30 cm. Find the scale of the plot and indicate through a sketch how a suitable scale can be constructed to read up to 1 m in the map.

Solution

$$\text{RF} = \frac{30}{1.2 \times 1{,}000 \times 100} = \frac{1}{4{,}000}$$

$$1 \text{ cm} = 4{,}000 \text{ cm} = 40 \text{ m}$$

Here, the maximum length to be shown is not given. Let us assume the length of the scale to be 15 cm.

$$15 \text{ cm} = 15 \times 40 = 600 \text{ m}$$

15 cm are divided into six parts, each representing 100 m. The left-hand division is further divided into 10 divisions horizontally at the base and 10 divisions vertically, and joined digonally to get a minimum reading 1 m.

So, the scale constructed is diagonal (Fig. P.1.16).

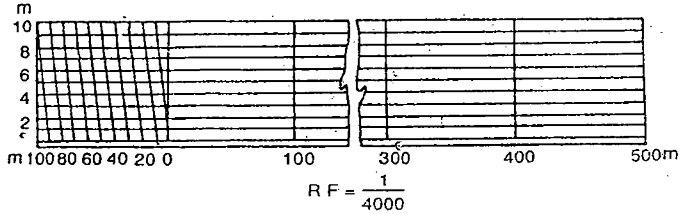


Fig. P .1.16

Problem 4 A rectangular plot of land of area 0.45 hectare is represented on a map by a similar rectangle are of 5 cm². Calculate the RF of the scale of the map. Draw a scale to read up to a single metre from the map. The scale should be long enough to measure up to 400 m (1 hectare = 10,000 m²).

Solution

$$5 \text{ cm}^2 = 0.45 \text{ hectare}$$

$$1 \text{ cm}^2 = \frac{0.45 \times 10,000}{5} \text{ m}^2 = 900 \text{ m}^2$$

$$1 \text{ cm} = 30 \text{ m}$$

$$RF = \frac{1}{3,000}$$

Maximum length to be shown = 400 m

$$\text{Length of scale} = \frac{1}{3,000} \times 400 \times 100 = 13.33 \text{ cm}$$

13.33 cm are divided into 4 parts, each representing 100 m. The left-hand-side division is again divided into 10 divisions horizontally at the base and 10 divisions vertically, and then joined diagonally to get 1 m.

Therefore, the scale constructed is diagonal (Fig. P.1.17).

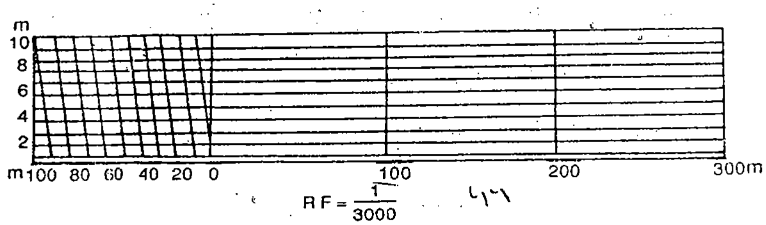


Fig. P .1.17

1.27 CONVERSION TABLE FOR IMPORTANT UNITS

Length

12 inches	= 1 foot	1 inch	= 2.54 cm
3 feet	= 1 yard	1 foot	= 0.3048 m

$5\frac{1}{2}$ yards	= 1 rod or pole	1 mile	= 1,760 yards
4 poles (66 ft)	= 1 chain		= 5,280 feet
10 chains	= 1 furlong		= 1.609 km
8 furlongs	= 1 mile	1 nautical mile	= 6,080 feet
6 feet	= 1 fathom	1 nautical mile	= 1.152 miles
120 fathoms	= 1 cable length	1 nautical mile	= 1.852 km

10 decametre	= 1 hectametre
10 hectametre	= 1 kilometre
1,000 metres	= 1 kilometre

Area

100 m²	= 1 are (a)	1 acre	= 4,840 square yards
100 ares	= 1 hectare		= 3.025 bighas
100 hectare	= 1 km²	1 bigha	= 1600 square yards
1 hectare	= 10,000 m²	20 kathas	= 1 bigha
640 acres	= 1 square mile	16 chattak	= 1 katha
1 hectare	= 2.47 acres	1 katha	= 720 square feet
484 square yards	= 1 square chain	1 chattak	= 45 square feet
10 square chain	= 1 acre		

SHORT QUESTIONS FOR VIVA

Q.1 What is the fundamental difference between surveying and levelling?

Ans. In surveying, the measurements are taken in the horizontal plane, but in levelling they are taken in the vertical plane.

Q.2 What is the fundamental difference between plane surveying and geodetic surveying?

Ans. In plane surveying, the curvature of the earth is not considered. But in geodetic surveying, the curvature of the earth is considered.

Q.3 What do you mean by the terms 'topographical map' and 'cadastral map'?

Ans. A map which shows the natural features of a country such as rivers, hills, roads, railways, villages, towns, etc. is known as a topographical map, and one which shows the boundaries of estates, fields, houses, etc. is known as a cadastral map.

Q.4 What is the main principle of surveying?

Ans. The fundamental principle of surveying is to work from the whole to the part.

Q.5 How is a chain folded and unfolded?

Ans. In order to fold the chain, a chainman moves forward by pulling the chain at the middle so that two halves come side by side. Then he places the pair of links on his left hand with his right hand until the two brass handles appear at the top.

To unfold the chain, a chainman holds the two brass handles in his left hand and throws the bunch with his right hand. Then one chainman stands at a station holding one handle and another chainman moves forward by holding the other handle.

Q.6 In a chaining operation, who is the leader and who the follower?

Ans. The chainman at the forward end of the chain who drags the chain is known as the leader.

The one at the rear end of the chain is known as the follower.

Q.7 While chaining a line, you have to measure through a steep sloping ground. What method should you apply?

Ans. The stepping method.

Q.8 Two stations are not intervisible due to intervening high ground. How will you range the line?

Ans. The ranging is to be done by the reciprocal method.

Q.9 What do you mean by normal tension?

Ans. The tension at which the measured distance is equal to the correct distance (i.e. when sag correction is neutralised by pull correction) is known as normal tension.

Q.10 What do you mean by RF?

Ans. The ratio of the distance on the drawing to the corresponding actual length of the object is known as RF.

Q.11 What is the difference between plain scale and diagonal scale?

Ans. The plain scale represents two successive units. The diagonal scale represents three successive units.

Q.12 What is hypotenusal allowance?

Ans. When one chain length is measured on sloping ground, then it shows a shorter distance on the horizontal plane. The difference between the sloping distance and horizontal distance is known as the hypotenusal allowance.

Q.13 How many ranging rods are required to range a line?

Ans. At least three ranging rods are required for direct ranging, and at least four for indirect ranging.

Q.14 What is the length of one link in a 20 m chain?

Ans. The 20 m chain is divided into 100 links. So, one link is 0.2 m, i.e. 20 cm, long.

EXERCISES

1. Classify surveying on the basis of instruments used and name all equipment necessary for field work involving any one of them.
2. Describe with sketches how an obstacle which interrupts chaining but not ranging can be crossed over in chain survey.
3. Describe briefly how plane surveying differs from geodetic surveying.
4. Describe the different types of chains used in survey indicating the relative advantages of each.
5. Describe how you will measure the distance between two stations on fairly level ground.
6. Describe with a sketch how you will measure distance on sloping ground.
7. Two stations A and B are not intervisible due to rising ground between them. Explain with a sketch how the line AB can be ranged if both the stations are visible from intermediate points.
8. What are the sources of error in chaining? What precautions would you take to guard against them?
9. The following slope distances were measured along a chain line with a 20 m steel tape. It was noted afterwards that the tape was 2.5 cm too long. Find the true total horizontal distance:

Slope distance (m)	*Difference in elevation between ends of tape* (m)
18.7	0.85
13.4	2.90
10.1	3.25
16.9	1.75

(Ans. 60.105 m)

10. A plan represents an area of 39,672 m^2 and measures 4.75 cm × 5.22 cm. Find the scale of the plot and indicate through a sketch how a suitable scale can be constructed to read up to 1 m on the plan.
11. A base line was measured to be 150 m long with a tape at a field temperature of 27°C, the applied pull being 14 kg. The tape was standardised at a temperature of 15°C with a pull of 8 kg. If the designated length of the tape is 20 m, weight of 1 cm^3 of tape = 7.86 g, total weight of tape = 0.8 kg, $E = 2.109 \times 10^6$ kg/cm^2 and coefficient of expansion of tape per degree Celsius = 11.2×10^{-6}, find the true length of the line. (Ans. 150.0081 m)
12. In chaining an area containing a pond, two points C and D are selected on either sides of chain station A such that A, C and D lie on a straight line. The point B which is on the other side of the pond is on the chain line AB. If distances AC, AD, BC and BD are 35, 45, 100, and 95 m respectively, determine the length of chain line AB and the angle which the inclined line CD makes with the chain lines AB. (Ans. AB = 89.43 m, angle = 88°11′)
13. A steel tape 20 m long, standardised at 15°C with a pull of 10 kg, was used to measure distance along a slope of 4°25′. If the mean temperature during measurement was 10°C, and the pull applied 16 kg, determine the correction required per tape length. Assume coefficient of expansion as 112×10^{-7} per °C, cross-sectional area of tape = 0.08 cm^2, $E = 2.1 \times 10^6$ kg/cm^2. (Ans. 0.0896 m too short)
14. To determine the width of a river, a chain line PQR was laid across it, the points Q and R being on two sides of river. From point S, 60 m from Q on line QS which was at right angles to PQ, the bearings of points R and P and were found to be 280° and 190° respectively. If the distance PQ was 32 m, determine the distance QR and draw the sketch. (Ans. QR = 112.51 m)
15. Choose the correct alternative for questions, (i) to (xviii).

(i) The object of surveying is to prepare a
(a) Drawing (b) Cross-section
(c) Map

(ii) The curvature of the earth is ignored in
(a) Geodetic surveying (b) Plane surveying
(c) Hydrographic surveying

(iii) The curvature of the earth is taken into account when the extent of area is more than
(a) 50 km^2 (b) 100 km^2
(c) 250 km^2

(iv) The main principle of surveying is to work from
(a) The centre to the boundary (b) The whole to the part
(c) The part to the whole

(v) Surveys which depict the natural features of a country are known as
(a) Cadastral surveys (b) Topographical surveys
(c) Engineering surveys

(vi) The diagonal scale is used to read
(a) One unit (b) Two units
(c) Three consecutive units

(vii) A 20 m chain is divided into
(a) 150 links (b) 100 links
(c) 200 links

(viii) A 30-m chain is divided into
(a) 100 links (b) 150 links
(c) 300 links

(ix) The length of Gunter's chain is
(a) 100 ft (b) 50 ft
(c) 66 ft

(x) For ranging a line, the number of ranging rods required is
(a) A least two (b) At least three
(c) At least four

(xi) The difference between the arc length and chord length for a distance of 18.2 km is only
(a) 5 cm (b) 10 cm
(c) 15 cm

(xii) Compensating error is proportional to
(a) L (b) $\sqrt{L}$
(c) L^2

(xiii) If θ be the angle of slope and l the sloping distance, slope correction is given by
(a) $l\,(1 - \sin\theta)$ (b) $l\,(1 - \cos\theta)$
(c) $l\,(1 - \sec\theta)$

(xiv) If l be the sloping distance and h the difference of level between two end points, slope correction is given by
(a) $\frac{h}{2l}$ (b) $\frac{h^2}{2l}$
(c) $\frac{2l}{h^2}$

(xv) If θ be the angle of slope and the length of chain be 30 m, then correction to be applied per chain length is
(a) $30\,(\cos\theta - 1)$ m (b) $30\,(\sec\theta - 1)$ m
(c) $30\,(\tan\theta - 1)$ m

(xvi) 'One link' means the distance from
(a) Centre to centre of middle rings (b) Centre to centre of inner rings
(c) Centre to centre of outer rings

(xvii) The end link is considered
(a) Including the length of the handle
(b) Excluding the length of the handle
(c) From the centre of the handle

(xviii) The walking step of a man is considered equal to
(a) 80 cm (b) 90 cm
(c) 100 cm

ANSWERS

Q.15	(i) (c)	(ii) (b)	(iii) (c)	(iv) (b)	(v) (b)
	(vi) (c)	(vii) (b)	(viii) (b)	(ix) (c)	(x) (b)
	(xi) (b)	(xii) (b)	(xiii) (b)	(xiv) (b)	(xv) (b)
	(xvi) (a)	(xvii) (a)	(xviii) (a)		

2

Chain Surveying

2.1 PRINCIPLE OF CHAIN SURVEYING

The principle of chain surveying is triangulation. This means that the area to be surveyed is divided into a number of small triangles which should be well conditioned. In chain surveying the sides of the triangles are measured directly on the field by chain or tape, and no angular measurements are taken. Here, the tie lines and check lines control the accuracy of work.

It should be noted that plotting triangles requires no angular measurements to be made, if the three sides are known.

Chain surveying is recommended when:

1. The ground surface is more or less level
2. A small area is to be surveyed
3. A small-scale map is to be prepared and
4. The formation of well-conditioned triangles is easy

Chain surveying is unsuitable when:

1. The area is crowded with many details
2. The area consists of too many undulations
3. The area is very large and
4. The formation of well-conditioned triangles becomes difficult due to obstacles

A. Large-Scale and Small-Scale Maps

When 1 cm of a map represents a small distance, it is said to be a large-scale map.

For example,

$$1 \text{ cm} = 1 \text{ m} \quad \text{i.e.} \quad RF = \frac{1}{100}$$

When 1 cm of the map represents a large distance, it is called a small-scale map.

For example,

$$1 \text{ cm} = 100 \text{ m} \quad \text{i.e.} \quad RF = \frac{1}{10{,}000}$$

A map having an RF of less than 1/500 is considered to be large-scale. A map of RF more than 1/500 is said to be small-scale.

2.2 WELL-CONDITIONED AND ILL-CONDITIONED TRIANGLES

A triangle is said to be well-conditioned when no angle in it is less than 30° or greater than 120°. An equilateral triangle is considered to be the best-condition or ideal triangle (Figs 2.1(a) and (b)).

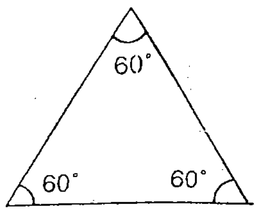


Fig. 2.1(a) Ideal Triangle

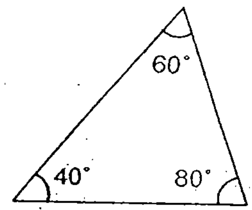


Fig. 2.1(b) Well-Conditioned Triangle

Well-conditioned triangles are preferred because their apex points are very sharp and can be located by a single 'dot'. In such a case, there is no possibility of relative displacement of the plotted point.

A triangle in which an angle is less than 30° or more than 120° is said to be ill-conditioned (Fig. 2.1(c)).

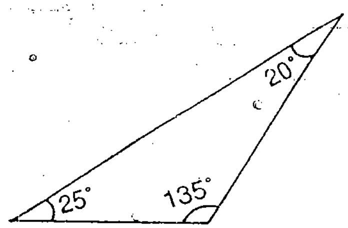


Fig. 2.1(c) Ill-Conditioned Triangle

Ill-conditioned triangles are not used in chain surveying. This is because their apex points are not sharp and well defined, which is why a slight displacement of these points may cause considerable error in plotting.

2.3 RECONNAISSANCE SURVEY AND INDEX SKETCH

Before the commencement of any survey work, the area to be surveyed is thoroughly examined by the surveyor, who then thinks about the possible arrangement of the framework of survey. This primary investigation of the area is termed as reconnaissance survey or reconnoitre.

During reconnaissance survey, the surveyor should walk over the area and note the various obstacles and whether or not the selected stations are intervisible. The main stations should be so selected that they enclose the whole area. The surveyor should also take care that the triangles formed are well-conditioned. He should note the various objects which are to be located.

The neat hand sketch of the area which is prepared during reconnaissance survey is known as the 'index sketch' or 'key plan'. The index sketch shows the skeleton of the survey work. It indicates the main survey stations, sub-stations, tie stations, base line, arrangement for framework of triangles and the approximate positions of different objects. This sketch is an important document for the surveyor and for the person who will plot the map. It should be attached to the starting page of the field book (Fig. 2.2).

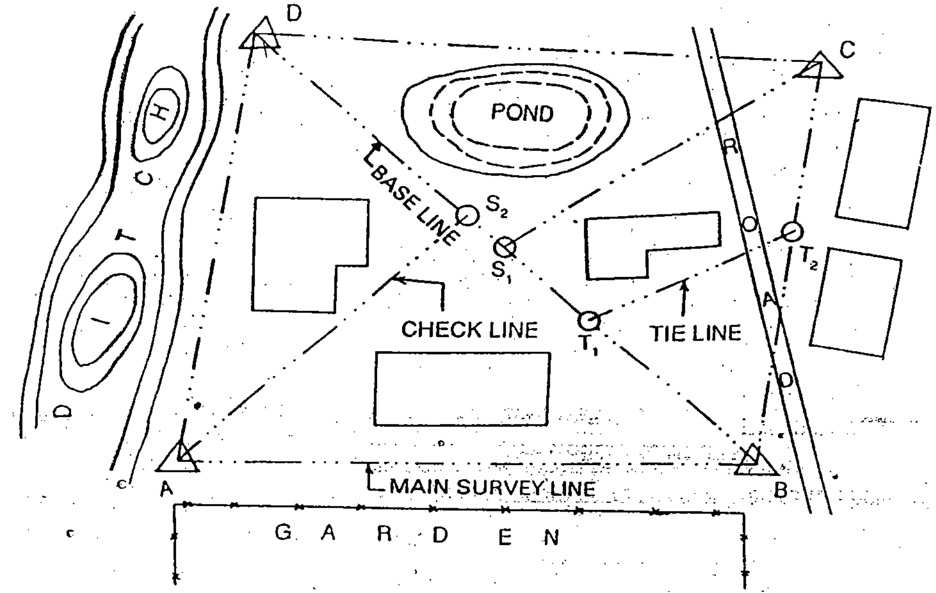


Fig. 2.2 Index Sketch

2.4 DEFINITIONS AND ILLUSTRATIONS

A. Survey Stations

Survey stations are the points at the beginning and the end of a chain line. They may also occur at any convenient points on the chain line. Such stations may be:

1. Main stations
2. Subsidiary stations and
3. Tie stations

1. Main stations Stations taken along the boundary of an area as controlling points are known as 'main stations'. The lines joining the main stations are called 'main survey lines'. The main survey lines should cover the whole area to be surveyed. The main stations are denoted by 'Δ' with letters A, B, C, D, etc. The chain lines are denoted by "— ... — ... — ... — ... —".

2. Subsidiary stations Stations which are on the main survey lines or any other survey lines are known as "subsidiary stations". These stations are taken to run subsidiary lines for dividing the area into triangles, for checking the accuracy of triangles and for locating interior details. These stations are denoted by '$\odot$' with letters S_1, S_2, S_3, etc.

3. Tie stations These are also subsidiary stations taken on the main survey lines. Lines joining the tie stations are known as tie lines. Tie lines are mainly taken to fix the directions of adjacent sides of the chain survey map. These are also taken to form 'chain angles' in chain traversing, when triangulation is not possible (chain angles are described in Chapter 3). Sometimes tie lines are taken to locate interior details. Tie stations are denoted by '⊙' with letters T_1, T_2, T_3, etc.

B. Base Line

The line on which the framework of the survey is built is known as the 'base line'. It is the most important line of the survey. Generally, the longest of the main survey lines is considered the base line. This line should be taken through fairly level ground, and should be measured very carefully and accurately. The magnetic bearings of the base line are taken to fix the north line of the map.

C. Check Line

The line joining the apex point of a triangle to some fixed point on its base is known as the 'check line'. It is taken to check the accuracy of the triangle. Sometimes this line helps to locate interior details.

D. Offset

The lateral measurement taken from an object to the chain line is known as 'offset'. Offsets are taken to locate objects with reference to the chain line. They may be of two kinds—perpendicular and oblique.

1. Perpendicular offsets When the lateral measurements are taken perpendicular to the chain line, they are known as perpendicular offsets (Fig. 2.3).

Perpendicular offsets may be taken in the following ways:

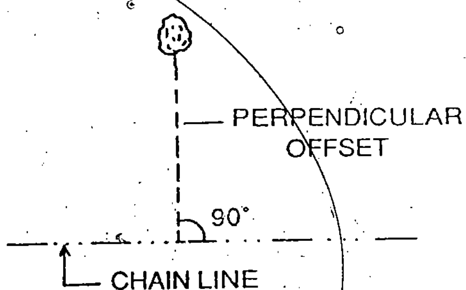


Fig. 2.3

(a) By setting a perpendicular by swinging a tape from the object to the chain line. The point of minimum reading on the tape will be the base of the perpendicular (Fig. 2.4).
(b) By setting a right angle in the ratio 3 : 4 : 5 (Fig. 2.5).
(c) By setting a right angle with the help of builder's square or tri-square (Fig. 2.6).
(d) By setting a right angle by cross-staff or optical square.

2. Oblique offsets Any offset not perpendicular to the chain line is said to be oblique. Oblique offsets are taken when the objects are at a long distance from the chain line or when it is not possible to set up a right angle due to some difficulties. Such offsets are taken in the following manner.

Suppose AB is a chain line and p is the corner of a building. Two points 'a' and 'b' are taken on the chain line. The chainages of 'a' and 'b' are noted. The

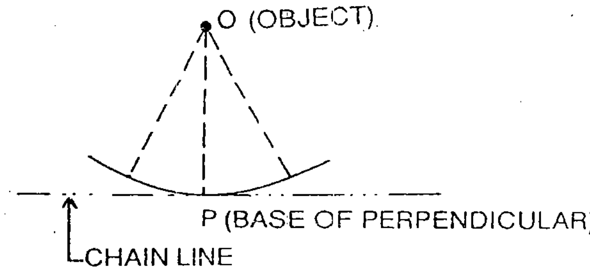


Fig. 2.4

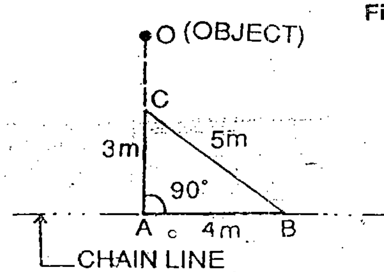


Fig. 2.5

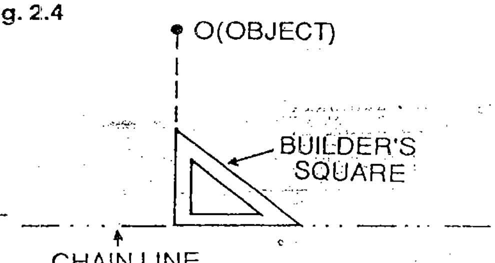


Fig. 2.6

distances 'ap' and 'bp' are measured and noted in the field book. Then 'ap' and 'bp' are the oblique offsets (Fig. 2.7). When the triangle abp is plotted, the apex point p will represent the position of the corner of the building.

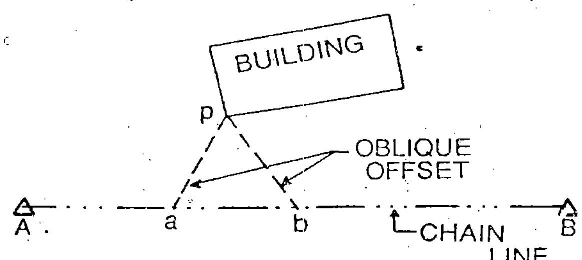


Fig. 2.7

Perpendicular offsets are preferred for the following reasons:

(a) They can be taken very quickly
(b) The progress of survey is not hampered
(c) The entry in the field book becomes easy
(d) The plotting of the offsets also becomes easy

3. Number of offsets The offsets should be taken according to the nature of the object. So, there is no hard and fast rule regarding the number of offsets. It should be remembered that the objects are to be correctly represented and hence the number of offsets should be decided on the field. Some guidelines are given below:

(a) When the boundary of the object is approximately parallel to the chain line, perpendicular offsets are taken at regular intervals (Fig. 2.8).
(b) When the boundary is straight, perpendicular offsets are taken at both ends of it (Fig. 2.9).

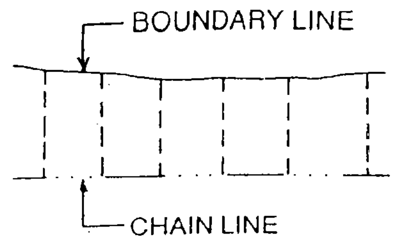


Fig. 2.8

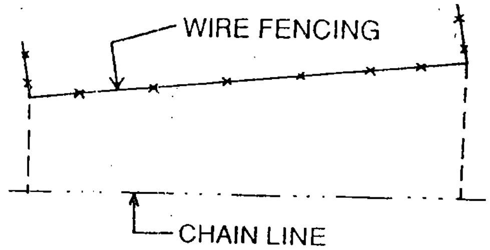


Fig. 2.9

(c) When the boundary line is zigzag, perpendicular offsets are taken at every point of bend to represent the shape of the boundary accurately. In such a case, the interval of the offsets may be irregular (Fig. 2.10).

(d) When a road crosses the chain line perpendicularly, the chainage of the intersection point is to be noted (Fig. 2.11).

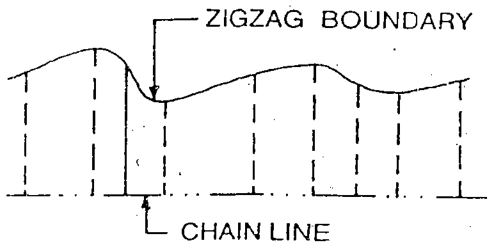


Fig. 2.10

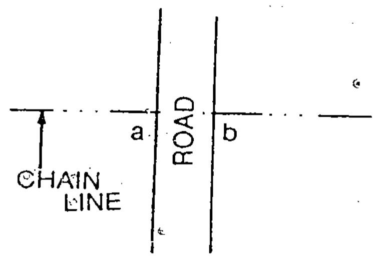


Fig. 2.11

(e) When a road crosses a chain line obliquely, the chainages of intersection points 'a' and 'b' are noted. Then at least one offset is taken on both sides of the inter-section points. More offsets may be taken depending on the nature of the road. Here, perpendicular offsets are taken at 'c' and 'd' (Fig. 2.12).

(f) When the building is small, its corners are fixed by perpendicular or oblique offsets and the other dimensions are taken directly on the field and noted in the field book (Fig. 2.13).

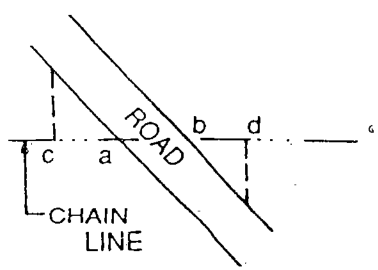


Fig. 2.12

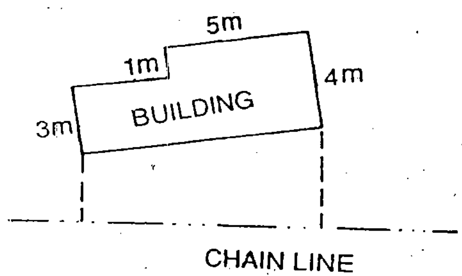


Fig. 2.13

(g) When the building is large, zigzag in shape and oblique to the chain line, then the corners are fixed by perpendicular or oblique offsets. Then the full plan of the building is drawn on a separate page along with all the dimensions. This page should be attached with the field book at the proper place (Fig. 2.14).

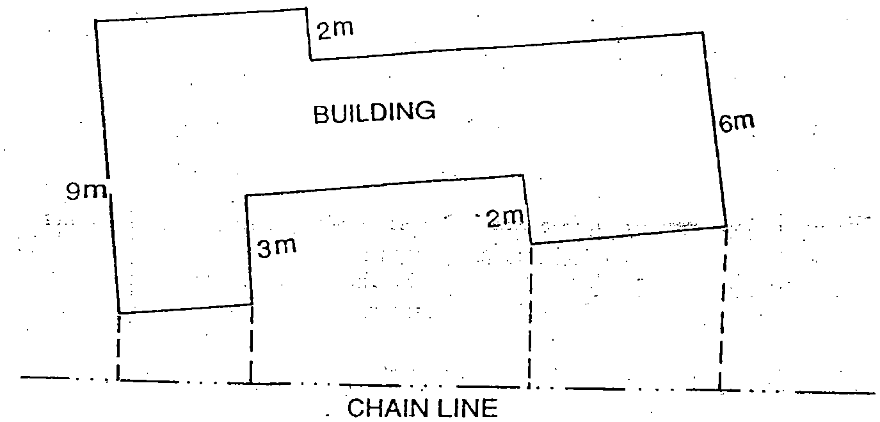


Fig. 2.14

(h) When the object is circular, perpendicular offsets are taken at short and regular intervals (Fig. 2.15).

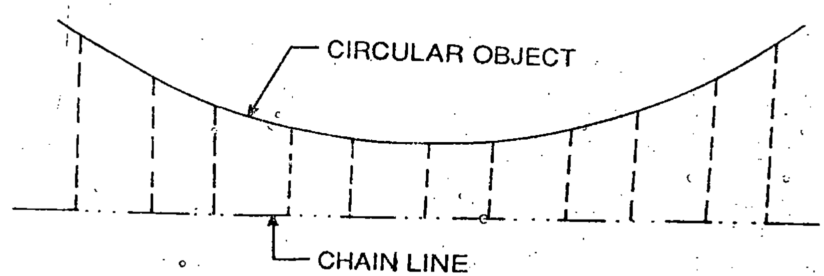


Fig. 2.15

4. Limiting length of offset The maximum length of the offset should not be more than the length of the tape used in the survey. Generally, the maximum length of offset is limited to 15 m. However, this length also depends upon the following factors:

(a) The desired accuracy of the map
(b) The scale of the map
(c) The maximum allowable deflection of the offset from its true direction and
(d) The nature of the ground

Problems on limiting length of offset

Problem 1 An offset was laid out 5° from its true direction and the scale of the map was 20 m to 1 cm. Find the maximum length of offset for the displacement of a point on the paper not to exceed 0.03 cm.

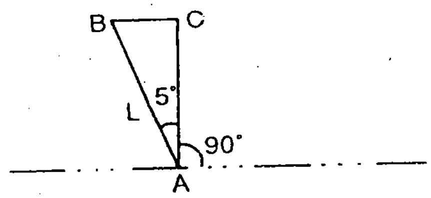


Fig. P.2.1

Solution Let AB be the actual length of offset which was laid out 5° from its true direction. So, BC is the displacement of the point.

Let the maximum length of offset, AB = L m

From triangle ABC, $\frac{BC}{AB} = \sin 5°$

or BC = AB sin 5° = L sin 5° m (displacement of the ground)

Since the scale is 1 cm to 20 m, 20 m on the ground represents 1 cm on the paper.

Therefore, L sin 5° on the ground represents $\frac{L \sin 5°}{20}$ cm on the paper.

According to the given condition, $\frac{L \sin 5°}{20} = 0.03$

$$L = \frac{0.03 \times 20}{\sin 5°} = 6.884 \text{ m}$$

Therefore, the maximum length of offset should be 6.884 m.

Problem 2 The length of the offset is 15 m and the scale of the plan 10 m to 1 cm. If the offset is laid out 3° from its true direction, find the displacement of the plotted point on the paper (i) perpendicular to the chain line, and (ii) parallel to the chain line.

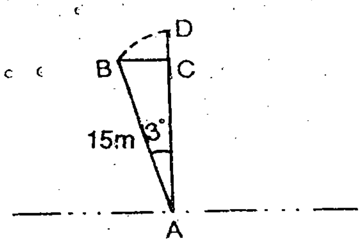


Fig. P.2.2

Solution Let AB be the actual length of offset, which is 15 m long and deflected by 3° from its true direction.

Here,

BC = Displacement parallel to chain line
CD = displacement perpendicular to chain line

(i) CD = AD – AC = AB – AC
= 15 – 15 cos 3°
= 15 (1 – cos 3°) m (displacement on the ground)

Since the scale is 1 cm to 10 m,
10 m on the ground = 1 cm on the map

$15(1 - \cos 3°)$ on the ground $= \frac{15(1 - \cos 3°)}{10}$

= 0.002 cm on the map

Required displacement perpendicular to chain line

= 0.002 cm (on paper)

(ii) BC = AB sin 3° = 15 sin 3° = 0.7850 m (displacement on ground)

Displacement parallel to chain line = $\frac{0.7850}{10}$ = 0.0785 cm (on paper)

E. Degree of Accuracy

Degree of accuracy is determined before the starting of any survey work. It is worked out according the following factors:

(a) Scale of plotting
(b) Permissible error in plotting

During reconnaissance survey, the length of the main survey lines are approximately determined by the pacing method. One pace or walking step of a man is considered to equal 80 cm. When the length of the survey lines or the extent of area to be surveyed is approximately known, the scale of the map may be assumed. Again, the permissible error in plotting may be obtained from the concerned department. Then the degree of accuracy in measurement is ascertained.

Let us now consider an example.

Suppose the scale of plotting is 5 m to 1 cm and the allowable error is 0.02 cm.

Then, 1 cm on the map = 500 cm on the ground

0.02 cm on the map = 500 × 0.02 = 10 cm on the ground

So, the measurement should be taken nearest to 10 cm.

2.5 SELECTION OF SURVEY STATIONS

The following points should be remembered during the selection of survey stations:

1. The stations should be so selected that the general principle of surveying may be strictly followed.
2. The stations should be intervisible.
3. The stations should be selected in such a way that well-conditioned triangles may be formed.
4. The base line should be the longest of the main survey lines.
5. The survey lines should be taken through fairly level ground, as far as practicable.

6. The main survey lines should pass close to the boundary line of the area to be surveyed.
7. The survey lines should be taken close to the objects so that they can be located by short offsets.
8. The tie stations should be suitably selected to fix the directions of adjacent sides.
9. The subsidiary stations should be suitably selected for taking check lines.
10. Stations should be so selected that obstacles to chaining are avoided as far as possible.
11. The survey lines should not be very close to main roads, as survey work may then be interrupted by traffic.

2.6 EQUIPMENTS FOR CHAIN SURVEY

The following equipments are required for conducting chain survey:

1.	Metric chain (20 m)	= 1 no.
2.	Arrows	= 10 nos
3.	Metallic tape (15 m)	= 1 no.
4.	Ranging rods	= 3 nos
5.	Offset rod	= 1 no.
6.	Clinometer	= 1 no.
7.	Plumb bob with thread	= 1 no.
8.	Cross staff or optical square	= 1 no.
9.	Prismatic compass with stand	= 1 no.
10.	Wooden pegs	= 10 nos
11.	Mallet	= 1 no.
12.	Field book	= 1 no.
13.	Good pencil	= 1 no.
14.	Pen knife	= 1 no.
15.	Eraser (rubber)	= 1 no.

2.7 THE FIELD BOOK

The notebook in which field measurements are noted is known as the 'field book'. The size of the field book is 20 cm × 12 cm and it opens lengthwise. Field books may be of two types:

1. Single-line, and
2. Double-line.

1. Single-line field book In this type of field book, a single red line is drawn through the middle of each page. This line represents the chain line, and the chainages are written on it. The offsets are recorded, with sketches, to the left or right of the chain line. The recording of the field book is started from the last page and continued towards the first page. The main stations are marked by '△' and subsidiary stations or tie stations are by '⊙' (Fig. 2.16).

2. Double-line field book In this type of field book, two red lines, 1.5 cm apart, are drawn through the middle of each page. This column represents the chain line, and the chainages are written in it. The offsets are recorded, with sketches, to the left or right of this column. The recording is begun from the last page and continued towards the first. The main stations are marked by '△' and subsidiary or tie stations by '⊙' (Fig. 2.17). This type of field book is commonly used.

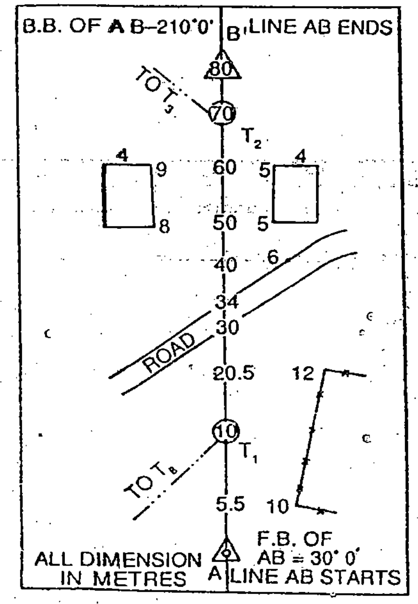


Fig. 2.16 Single-Line Field Book

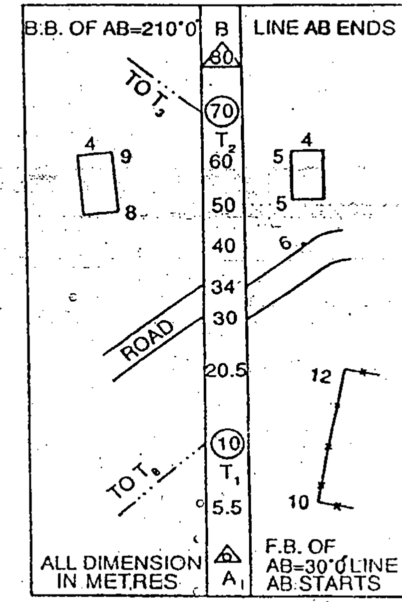


Fig. 2.17 Double-Line Field Book

A. Problems on Entering Records in Field Book

Problem 1 While measuring a chain line AB, the following offsets are taken. How would you enter the field book?

(a) A telegraph post is 10 m perpendicularly from chainage 2.5 m to the right of the chain line.
(b) A road crosses obliquely from left to right at chainage 10 m and 14 m. Perpendicular offsets are 2 m and 3 m to the side of the road from chainage 5 m and 20 m respectively.
(c) A tube-well is 5 m perpendicularly from chainage 30 m to the left of the chain line.
(d) Total chainage of AB is 45 m.

Solution

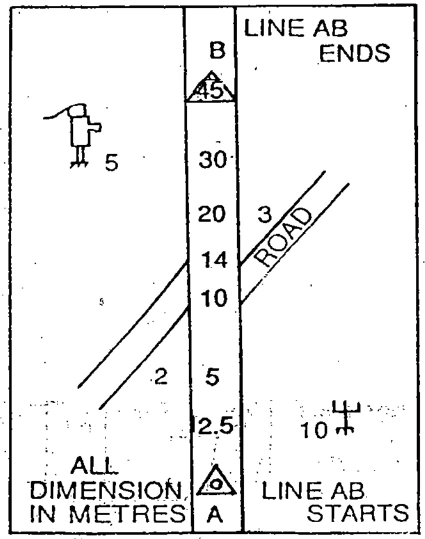


Fig. P.2.3

Problem 2 The base line AC of a chain survey is measured and the following records are noted. Make the necessary entries in a field book.

(a) The corners of a building are 9 and 9.5 m from chainage 7.5 and 18 m to the left of the chain line. The building is 7 m wide.
(b) A 4 m wide road runs about parallel to the right of the chain line. Offsets are 2, 2.1, 2.2, and 2.15 m at chainages 0, 20, 40, and 55.5 m respectively.
(c) A check line is taken from the sub-station at chainage 25 m to the left.
(d) The total chainage of the base line is 55.5 m.
(e) The fore bearing and back bearing of the base line are 30°30′ and 210°30′ respectively.

Solution

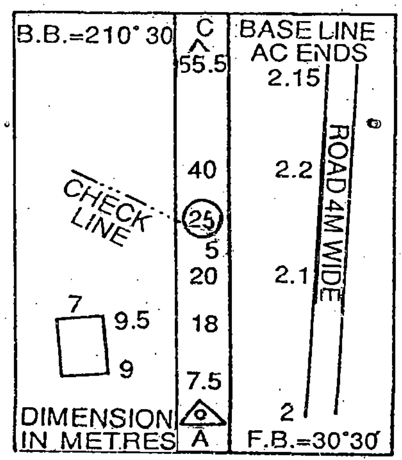


Fig. P.2.4

Problem 3 Enter the field book according to the following field notes:

(a) Chainage of line AB is 95.5 m
(b) The offsets to the pond at the left of chain line are as follows:
Chainage—10, 15, 20, 25, 30 m
Offset —16, 12, 10, 14, 20 m
(c) The offsets to the river at the right of the chain line are:
Chainage—5, 25, 40, 80 m
Offset —13, 17, 19, 19.5 m

Solution

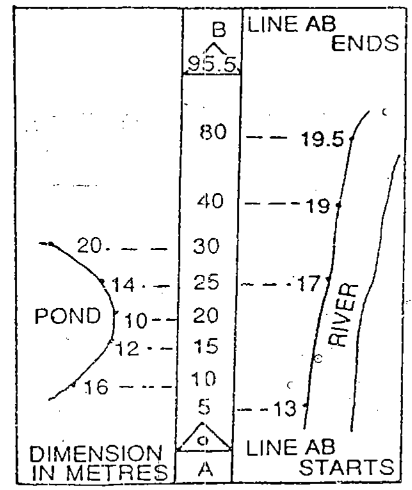


Fig. P.2.5

B. Precautions to be Taken While Entering the Field Book

1. All measurements should be noted as soon as they are taken.
2. Each chain line should be recorded on a separate page. Normally it should start from the bottom of one page and end on the top of another. No line should be started from any intermediate position.
3. Over-writing should be avoided.
4. Figures and hand-writing should be neat and legible.
5. Index-sketch, object-sketch and notes should be clear.
6. Reference sketches should be given in the field book, so that the station can be located when required.
7. The field book should be entered in pencil and not in ink.
8. If an entry is incorrect or a page damaged, cancel the page and start the entry from a new one.
9. Erasing a sketch, measurement or note should be avoided.
10. The surveyor should face the direction of chaining so that the left-hand and right-hand objects can be recorded without any confusion.
11. The field-book should be carefully preserved.

12. The field-book should contain the following: (i) name, (ii) location, and (iii) date, of survey; (iv) name of party members, and (v) page index of chain line.

2.8 PROCEDURE OF FIELD WORK

Field work of chain survey should be carried out according to the following steps:

1. Reconnaissance Before starting survey work, the surveyor should walk over the whole area to be surveyed in order to examine the ground and determine the possible arrangement of framework of survey. During this investigation, he should examine the intervisibility of the main survey stations. He should ensure that the whole area is enclosed by main survey lines, and also that it is possible to form well-conditioned triangles. He should observe various objects and boundary lines carefully and select the survey lines in such a manner that the objects can be located by short offsets. The base line should preferably be taken through the centre of the area and on fairly level ground.

2. Index sketch After preliminary inspection of the area, the surveyor should prepare a neat hand sketch showing the arrangement of the framework and approximate position of the objects. He should note the names of the stations on the sketch maintaining some order (clockwise or anticlockwise). The field work should be executed according to this index sketch. The names and sequence of chain lines should be followed as directed in the index sketch (Fig. 2.18). The 'base line' should be clearly indicated in the index sketch.

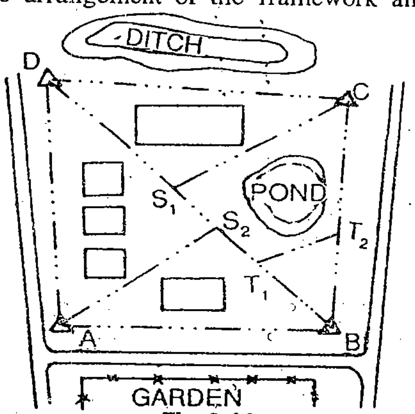


Fig. 2.18

3. Marking the stations on the ground After reconnaissance, the stations are marked on the ground by wooden pegs. These pegs are generally 2.5 cm square and 15 cm long, and have pointed ends. They are driven into the ground firmly, and there should be a height of 2.5 cm above the ground. The station point is marked with a cross so that it can be traced if the wooden peg is removed by somebody (Fig. 2.19).

4. Reference sketches To take precautions against station pegs being removed or missed, a reference sketch should be made for all main stations. It is nothing but a hand sketch of the station showing at least two measurements from some permanent objects. A third measurement may also be taken (Fig. 2.20).

5. Taking measurements of survey lines and noting them in the field book Ranging and chaining is started from the base line, which should be measured carefully. The magnetic bearings of the base line are measured by prismatic compass. These

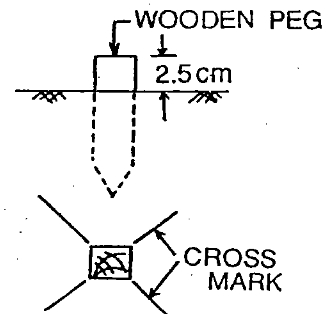


Fig. 2.19

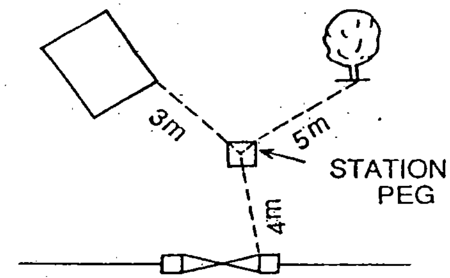
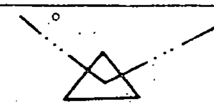
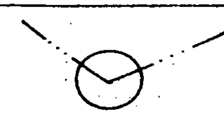
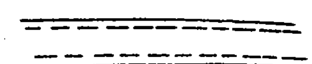


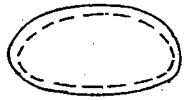
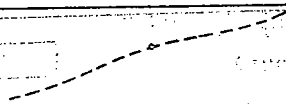
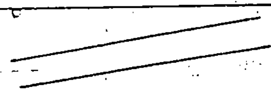
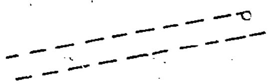
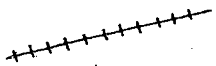
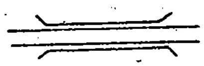
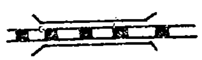
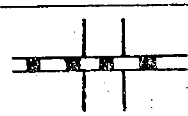
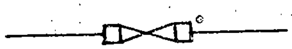
Fig. 2.20

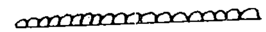
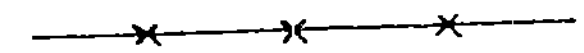
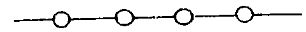
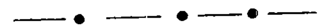
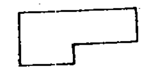
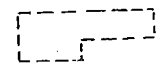
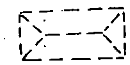
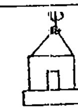
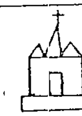
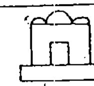
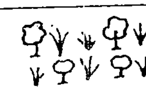
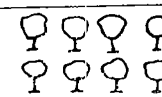
measurements are noted in the field book showing the offsets to the left or right according to their position. Then the other survey lines are ranged and chained maintaining the sequence of the traverse (i.e. AB, BC, CD etc). The offsets and other field records are noted simultaneously. The check lines and tie lines are also measured and noted at the proper place. The station marks are preserved carefully until field work is completed.

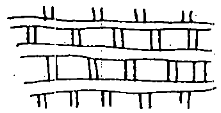
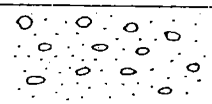
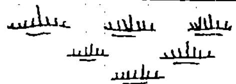
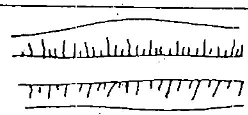
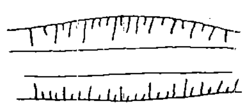
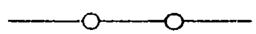
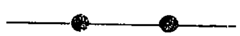
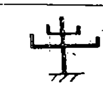
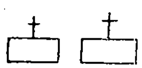
2.9 CONVENTIONAL SYMBOLS

In a map the objects are shown by symbols and not by names. So the surveyor should know the following standard conventional symbols for some common objects.

S. No.	Object	Symbol	Colour
1.	North line		Black
2.	Main stations or triangulation stations		Red or crimson lake
3.	Traverse stations or substations		Red or crimson lake
4.	Chain line		Red or crimson lake
5.	River		Prussian blue
6.	Canal		Prussian blue

S. No.	Object	Symbol	Colour
7.	Lake or pond		Prussian blue
8.	Open well		Prussian blue
9.	Tube well		Black
10.	Footpath		Black
11.	Metalled road		Burnt sienna
12.	Unmetalled road		Burnt sienna
13.	Railway line (single)		Black
14.	Railway line (double)		Black
15.	Road bridge or culvert		Black
16.	Railway bridge or culvert		Black
17.	Level-crossing		Black and burnt sienna
18.	Wall with gate		Black
19.	Boundary line		Black

S. No.	Object	Symbol	Colour
20.	Hedge		Green
21.	Wire fencing		Black
22.	Pipe fencing		Prussian blue
23.	Wood fencing		Yellow
24.	Building (*Pukka*)		Crimson lake
25.	Building (*katcha*)		Umber
26.	Huts		Yellow
27.	Temple		Crimson lake
28.	Church		Crimson lake
29.	Mosque		Crimson lake
30.	Benchmark		Black
31.	Tree		Green
32.	Jungle		Green
33.	Orchard		Green

S. No.	Object	Symbol	Colour
34.	Cultivated land		Black and green
35.	Barren land		Black
36.	Rough pasture		Black
37.	Marsh or swamp		Black
38.	Embankment		Black
39.	Cutting		Black
40.	(a) Telegraph line		Black
	(b) Telegraph post		Black
41.	(a) Electric line		Black
	(b) Electric post		Black
42.	Burial ground or cemetery	 OR 	Crimson lake

2.10 EQUIPMENTS FOR PLOTTING

1. Drawing board (normal size—1,000 mm × 700 mm)
2. Tee-square
3. Set-square (45° and 60°)

4. Protractor
5. Cardboard scale—set of eight
6. Instrument box
7. French curve
8. Offset scale
9. Drawing paper of good quality (normal size—880mm × 625 mm)
10. Pencils of good quality—2 H, 3 H or 4 H
11. Eraser (rubber) of good quality
12. Board clips or pins
13. Ink (Chinese ink or Indian ink) of required shade
14. Colour of required shade
15. Inking pen (or Hi-tech pen) and brushes
16. Handkerchief, knife, paperweight, etc.
17. Mini drafter

2.11 PROCEDURE OF PLOTTING

1. A suitable scale is chosen so that the area can be accommodated in the space available on the map.
2. A margin of about 2 cm from the edge of the sheet is drawn around the sheet.
3. The title block is prepared on the right hand bottom corner.
4. The north line is marked on the right-hand top corner, and should preferably be vertical. When it is not convenient to have a vertical north line, it may be inclined to accommodate the whole area within the map.
5. A suitable position for the base line is selected on the sheet so that the whole area along with all the objects it contains can be drawn within the space available in the map.
6. The framework is completed with all survey lines, check lines and tie lines. If there is some plotting error which exceeds the permissible limit, the incorrect lines should be resurveyed.
7. Until the framework is completed in proper form, the offsets should not be plotted (Fig. 2.21).
8. The plotting of offsets should be continued according to the sequence maintained in the field book.
9. The main stations, substations, chain line, objects, etc. should be shown as per standard symbols (Sec. 2.10).
10. The conventional symbols used in the map should be shown on the right-hand side.
11. The scale of the map is drawn below the heading or in some suitable space. The heading should be written on the top of the map.
12. Unnecessary lines, objects, etc. should be erased.
13. The map should not contain any dimensions.

Inking of the map The inking of the map should be started after the pencil work is completed in all respects. The colour conventions are shown in Sec. 2.10.

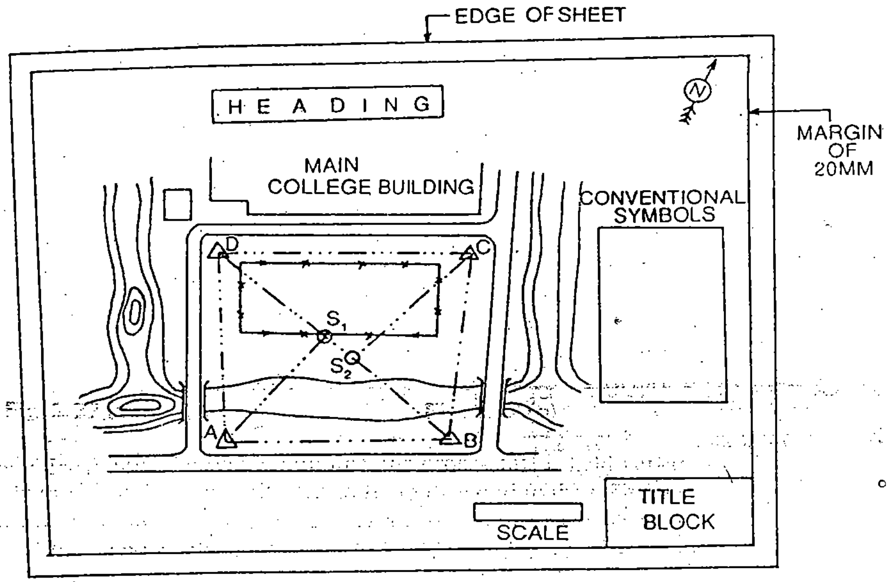


Fig. 2.21 Sample Plotting of Map Showing Different Positions

The inking should be begun from the left-hand-side towards the right-hand-side, and from the top towards the bottom.

Colouring of the map In general, colour washing of engineering survey maps is not recommended. However, if it is necessary, the colour shades should be very light, and according to the colour conventions shown in Sec. 2.10. The colouring should also be started from the left-hand-side towards the right and from the top towards the bottom.

2.12 CROSS-STAFF AND OPTICAL SQUARE

A. Cross-staff

The cross-staff is a simple instrument for setting out right angles. There are three types of cross-staves.

1. Open
2. French
3. Adjustable

The open cross-staff is commonly used.

Open cross-staff The open cross-staff consists of four metal arms with vertical slits. The two pairs of arms (AB and BC) are at right angles to each other. The vertical slits are meant for sighting the object and the ranging rods. The cross-staff is mounted on a wooden pole of length 1.5 m and diameter 2.5 cm. The pole is fitted with an iron shoe. (Fig. 2.22(a)).

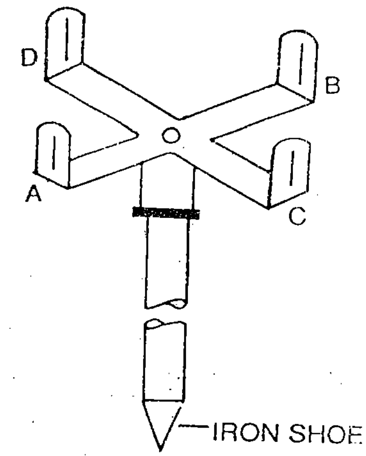


Fig. 2.22 (a)

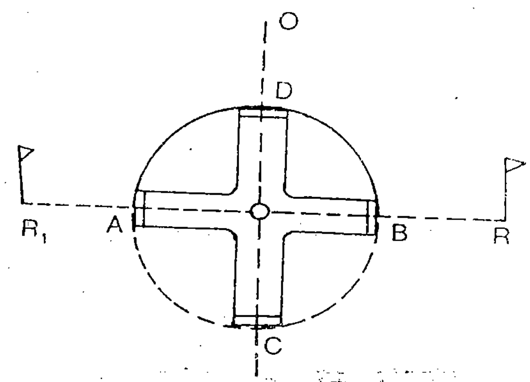


Fig. 2.22 (b)

For setting out a perpendicular on a chain line, the cross-staff is held vertically at the approximate position. Suppose slits A and B are directed to the ranging rods (R, R_1) fixed at the end stations. Slits C and D are directed to the object (O). Looking through slits A and B, the ranging rods are bisected. At the same time, looking through slits C and D, the object O is also bisected. To bisect the object and the ranging rods simultaneously, the cross staff may be moved forward or backward along the chain line (Fig. 2.22(b)).

B. Optical Square

An optical square is also used for setting out right angles. It consists of a small circular metal box of diameter 5 cm and depth 1.25 cm. It has a metal cover which slides round the box to cover the slits. The following are the internal arrangements of the optical square.

1. A horizon glass H is fixed at the bottom of the metal box. The lower half of the glass is unsilvered and the upper half is silvered.
2. An index glass I is also fixed at the bottom of the box which is completely silvered.
3. The angle between the index glass and horizon glass is maintained at 45°.
4. The opening 'e' is a pinhole for eye E, 'b' is a small rectangular hole for ranging rod B, 'P' is a large rectangular hole for object P.
5. The line EB is known as horizon sight and IP as index sight.
6. The horizon glass is placed at an angle of 120° with the horizon sight. The index glass is placed at an angle of 105° with the index sight.
7. The ray of light from P is first reflected from I, then it is further reflected from H, after which it ultimately reaches the eye E (Fig. 2.23(a)).

Principle According to the principle of reflecting surfaces, the angle between the first incident ray and the last reflected ray is twice the angle between the mirrors. In this case, the angle between the mirrors is fixed at 45°. So, the angle between the horizon sight and index sight will be 90°.

Setting up the perpendicular by optical square

1. The observer should stand on the chain line and approximately at the position where the perpendicular is to be set up.
2. The optical square is held by the arm at the eye level. The ranging rod at the forward station B is observed through the unsilvered portion on the lower part of the horizon glass.
3. Then the observer looks through the upper silvered portion of the horizon glass to see the image of the object P.
4. Suppose the observer finds that the ranging rod B and the image of object P do not coincide. Then he should move forward or backward along the chain line until the ranging rod B and the image of P exactly coincide (Figs 2.23(b) and 2.23(c)).
5. At this position the observer marks a point on the ground to locate the foot of the perpendicular.

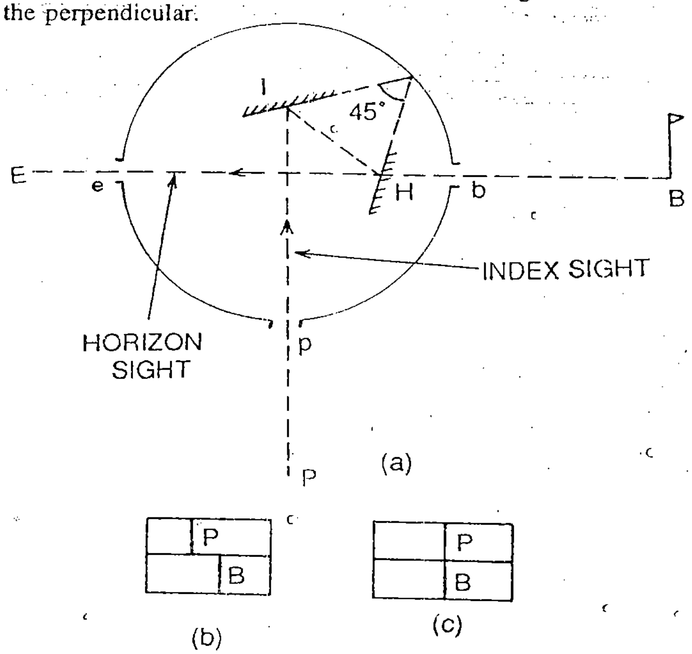


Fig. 2.23

SHORT QUESTIONS WITH ANSWERS FOR VIVA

Q. 1 What is the principle of chain surveying?
Ans. The principle of chain surveying is triangulation.
Q. 2 What do you mean by triangulation?
Ans. The method of dividing an area into a number of triangles is known as triangulation.
Q. 3 Why is the triangle preferred to the quadrilateral?
Ans. The triangle is preferred just it is a simple figure which can be drawn by just knowing the lengths of its sides.

Q. 4 What is the disadvantage of using ill-conditioned triangles?
Ans. The apex points of an ill-conditioned triangle are not well defined and sharp. This may cause some confusion while marking the actual point correctly on the map.
Q. 5 What is reconnaissance survey?
Ans. The preliminary inspection of the area to be surveyed is known as reconnaissance survey.
Q. 6 What is an index sketch?
Ans. During reconnaissance survey, a neat hand sketch is prepared showing the framework of the survey. This sketch is known as the index sketch.
Q. 7 What is 'base line of survey'?
Ans. The base line is the backbone of the survey. The framework of the survey is prepared on this line.
Q. 8 How is the north line of the chain survey map fixed?
Ans. The north line of the chain survey map is fixed by taking the magnetic bearings of the base line by prismatic compass.
Q. 9 Suppose you are asked to conduct a chain survey in a crowded town. What would you say?
Ans. In chain survey, the whole area is to be divided into a number of triangles. But the formation of triangles is not possible in a crowded town. So, I would reject the proposal.
Q. 10 What should be the maximum length of offset?
Ans. The maximum length of offset should be within the length of the tape used. Generally, it should not be more than 15 m.
Q. 11 How is a station marked on the ground?
Ans. The station is marked on the ground by a wooden peg, and with a cross on the station point.
Q. 12 What is the need of a reference sketch?
Ans. If the station peg is removed by someone, the station can be located accurately with the help of the measurements shown in the reference sketch.
Q. 13 How will you set up a perpendicular with the help of only a chain and a tape?
Ans. By forming a triangle in the ratio 3 : 4 : 5 using the chain and tape.
Q. 14 Who are the 'leader' and 'follower' when a line is being chained?
Ans. The chain man at the forward end of the chain who drags the chain is known as the 'leader'. The one at the rear end of the chain who holds the 'zero' end at the station is known as the 'follower'
Q. 15 Why does the field book open lengthwise?
Ans. If the field book is opened lengthwise, it become easy to maintain the continuation of a chain line.
Q. 16 Why is the scale always drawn in the map?
Ans. The paper on which the map is drawn may shrink or expand due to various reasons. If the scale is plotted on the map, then it is also reduced or enlarged proportionately. So, the distances on the map measured by this scale remain unaltered.
Q. 17 What is it necessary to provide tallies in a chain?
Ans. Tallies are provided in a chain for the facility of counting some fractional length of the chain, when the full chain length is not required.
Q. 18 What do you mean by the term 'ideal triangle'?
Ans. An equilateral triangle is said to be ideal.

EXERCISES

1. (a) What is the principle of chain surveying?

(b) When is chain survey recommended?
(c) When a chain survey becomes inconvenient?

2. (a) What is a well-conditioned triangle?
Explain clearly why it is preferred to an ill-conditioned triangle.
(b) The sides of a triangle are 156, 103 and 257 m long. Examine whether the triangle is well-conditioned.
3. Explain, with a neat sketch, the principle and use of an optical square.
4. What are offsets? How are perpendicular and oblique offsets taken?
5. (a) What should be the limiting length of offset?
(b) Explain, with sketches, how the number of offsets are decided for some common objects.
6. Describe, with a sketch, the construction and use of an open cross-staff.
7. What is a field book? What kind of field book would you prefer and why?
8. Describe the field procedure of chain survey.
9. (a) What are the points to be kept in mind while selecting survey stations?
(b) What precautions should be taken while taking field notes?
10. Give the conventional symbols for the following:
River, swamp, pond, orchard, rough pasture, embankment, cutting, cultivated land, barren land.
11. Select the correct alternatives for questions (i) to (xv).
(i) Chain survey is recommended when the area is
(a) Crowded (b) Undulating (c) Simple and fairly level
(ii) In chain survey the area is divided into
(a) Rectangles (b) Triangles (c) Squares
(iii) The sketch prepared during reconnaissance survey is known as the
(a) Hand sketch (b) Index sketch (c) Rough sketch
(iv) A triangle is said to be well-conditioned when its angles should lie between
(a) 30° and 120° (b) 20° and 150° (c) 15° and 135°
(v) The working principle of the optical square is based on
(a) Reflection (b) Refraction (c) Double reflection
(vi) The field records of the chain survey is entered in a/an
(a) Exercise book (b) Field book (c) Level book
(vii) The chain man who drags the chain is called the
(a) Captain (b) Leader (c) Follower
(xiii) The preliminary inspection of the area to be surveyed is known as
(a) Primary survey (b) Reconnaissance survey
(c) Route survey
(ix) The limiting length of offset depends upon the
(a) Scale of plotting (b) Method of measurement
(c) Method of layout
(x) The main survey stations are located on the ground by
(a) Index sketches (b) Reference sketches (c) Line sketch
(xi) In an optical square, the mirrors are fixed at an angle of
(a) 30° (b) 60° (c) 45°
(xii) Perpendicular offsets may be taken by setting the right angle in the ratio
(a) 3 : 6 : 9 (b) 3 : 4 : 5 (c) 2 : 4 : 5
(xiii) For taking an oblique offset which makes an angle of 45° with the chain line, the instrument used is the
(a) Adjustable cross-staff (b) Open cross-staff
(c) French cross-staff
(xiv) If a 20 m chain gets displaced from correct alignment by a perpendicular distance d m, then the error is given by
(a) $\frac{d^2}{40}$ (b) $\frac{d^2}{60}$ (c) $\frac{d^2}{80}$
(xv) If a wooded area obstructs the chain line, then it is crossed by the
(a) Profile line (b) Random line (c) Projection line

ANSWERS

11. (i) c	(ii) b	(iii) b	(iv) a	(v) c
(vi) b	(vii) b	(viii) b	(ix) a	(x) b
(xi) c	(xii) b	(xiii) c	(xiv) a	(xv) b

3

Compass Traversing

3.1 INTRODUCTION AND PURPOSE

In chain surveying, the area to be surveyed is divided into a number of triangles. This method is suitable for fairly level ground covering small areas. But when the area is large, undulating and crowded with many details, triangulation (which is the principle of chain survey) is not possible. In such an area, the method of traversing is adopted.

In traversing, the framework consists of a number of connected lines. The lengths are measured by chain or tape and the directions identified by angle measuring instruments. In one of the methods, the angle measuring instrument used is the compass. Hence, the process is known as compass traversing.

Note: Consideration of the traverse in an anticlockwise direction is always convenient in running the survey lines.

3.2 DEFINITIONS

1. True meridian The line or plane passing through the geographical north pole, geographical south pole and any point on the surface of the earth, is known as the 'true meridian' or 'geographical meridian'. The true meridian at a station is constant. The true meridians passing through different points on the earth's surface are not parallel, but converge towards the poles. But for surveys in small areas, the true meridians passing through different points are assumed parallel.

The angle between the true meridian and a line is known as 'true bearing' of the line. It is also known as the 'azimuth' (Fig. 3.1).

2. Magnetic meridian When a magnetic needle is suspended freely and balanced properly, unaffected by magnetic substances, it indicates a direction. This direction is known as the 'magnetic meridian'.

The angle between the magnetic meridian and a line is known as the 'magnetic bearing' or simply the 'bearing' of the line (Fig. 3.1).

3. Arbitrary meridian Sometimes for the survey of a small area, a convenient direction is assumed as a meridian, known as the 'arbitrary meridian'. Sometimes the starting line of a survey is taken as the arbitrary meridian.

The angle between the arbitrary meridian and a line is known as the 'arbitrary bearing' of the line.

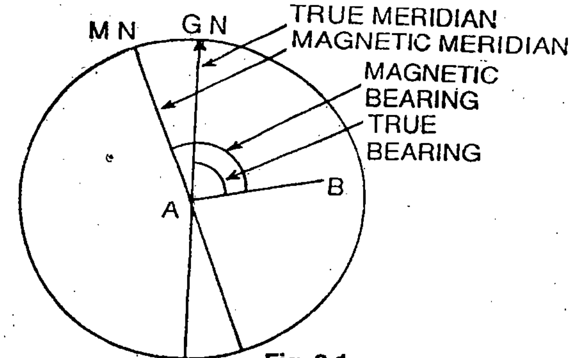


Fig. 3.1

4. Grid meridian Sometimes, for preparing a map some state agencies assume several lines parallel to the true meridian for a particular zone. These lines are termed as 'grid lines' and the central line the 'grid meridian'. The bearing of a line with respect to the grid meridian is known as the 'grid bearing' of the line.

5. Designation of magnetic bearing Magnetic bearings are designated by two systems:

(i) Whole circle bearing (WCB), and
(ii) Quadrantal bearing (QB).

(a) Whole Circle Bearing (WCB) The magnetic bearing of a line measured clockwise from the north pole towards the line, is known as the 'whole circle bearing, of that line. Such a bearing may have any value between 0° and 360°. The whole circle bearing of a line is obtained by prismatic compass (Fig. 3.2).

For example, in Fig. 3.2,

WCB of AB = θ_1
WCB of AC = θ_2
WCB of AD = θ_3
WCB of AE = θ_4

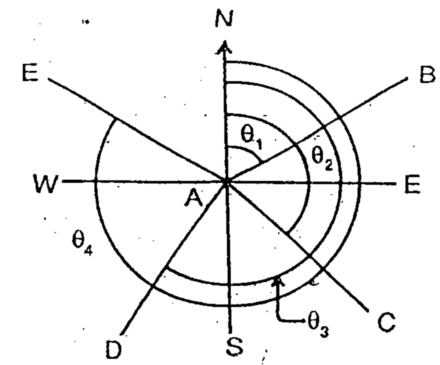


Fig. 3.2

(b) Quadrantal Bearing (QB) The magnetic bearing of a line measured clockwise or counterclockwise from the North Pole or South Pole (whichever is nearer the line) towards the East or West, is known as the 'quadrantal bearing' of the line. This system consists of four quadrants—NE, SE, SW and NW. The value of a quadrantal bearing lies between 0° and 90°, but the quadrants should always be mentioned. Quadrantal bearings are obtained by the surveyor's compass (Fig. 3.3).

For example,
QB of AB = $N\theta_1$ E
QB of AC = $S\theta_2$ E
QB of AD = $S\theta_3$ W
QB of AE = $N\theta_4$ W

6. Reduced bearing (RB) When the whole circle bearing of a line is converted to quadrantal bearing, it is termed the 'reduced bearing'. Thus, the reduced bearing is similar to the quadrantal bearing. Its value lies between 0° and 90°, but the quadrants should be mentioned for proper designation.

The following table should be remembered for conversion of WCB to RB:

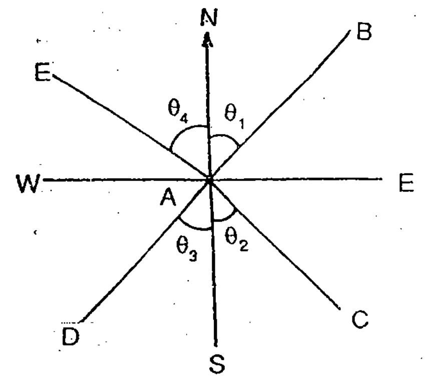


Fig. 3.3

WCB between	Corresponding RB	Quadrant
0° and 90°	RB = WCB	NE
90° and 180°	RB = 180° – WCB	SE
180° and 270°	RB = WCB – 180°	SW
270° and 360°	RB = 360° – WCB	NW

7. Fore and back bearing The bearing of a line measured in the direction of the progress of survey is called the 'fore bearing' (FB) of the line.

The bearing of a line measured in the direction opposite to the survey is called the 'back bearing' (BB) of the line (Fig. 3.4).

For example, in Fig. 3.4(a), FB of AB = θ

BB of AB = θ_1

In Fig. 3.4(b), FB of BA = θ

BB of BA = θ_1

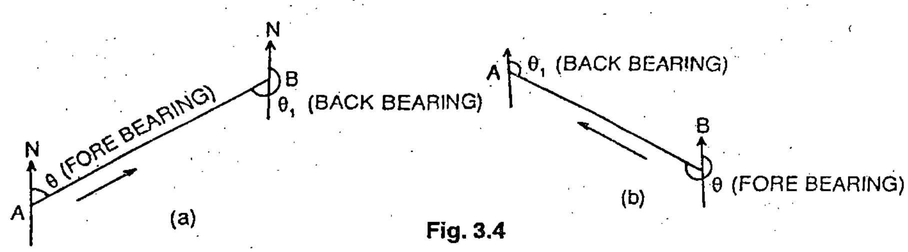


Fig. 3.4

Remember the following:

(a) In the WCB system, the difference between the FB and BB should be exactly 180°. Remember the following relation:

$$BB = FB \pm 180°$$

Use the positive sign when FB is less than 180°, and the negative sign when it is more than 180°.

(b) In the quandrantal bearing (i.e. reduced bearing) system, the FB and BB are numerically equal but the quadrants are just opposite.

For example, if the FB of AB is N 30° E, then its BB is S 30° W.

8. Magnetic declination The horizontal angle between the magnetic meridian and true meridian is known as 'magnetic declination'.

When the north end of the magnetic needle is pointed towards the west side of the true meridian, the position is termed 'Declination West' (θ W).

When the north end of the magnetic needle is pointed towards the east side of the true meridian, the position is termed 'Declination East' (θ E) (Fig. 3.5).

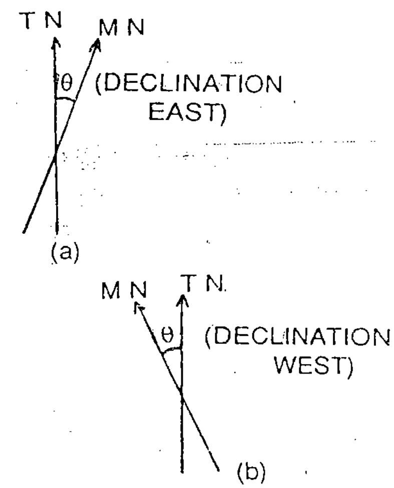


Fig. 3.5

9. Isogonic and agonic lines Lines passing through points of equal declination are known as 'isogonic' lines.

The line passing through points of zero declination is said to be the 'agonic' line (Fig. 3.6).

The Survey of India Department has prepared a map of India in which the isogonic and agonic lines are shown properly as a guideline to conduct the compass survey in different parts of the country.

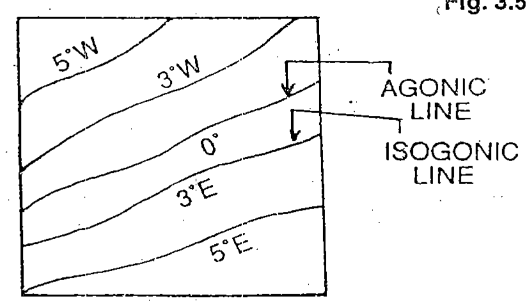


Fig. 3.6

10. Variation of magnetic declination The magnetic declination at a place is not constant. It varies due to the following reasons:

(a) Secular Variation The magnetic meridian behaves like a pendulum with respect to the true meridian. After every 100 years or so, it swings from one direction to the opposite direction, and hence the declination varies. This variation is known as 'secular variation'.

(b) Annual Variation The magnetic declination varies due to the rotation of the earth, with its axis inclined, in an elliptical path around the sun during a year. This variation is known as 'annual variation'. The amount of variation is about 1 to 2 minutes.

(c) Diurnal Variation The magnetic declination varies due to the rotation of the earth on its own axis in 24 hours. This variation is known as 'diurnal variation'. The amount of variation is found to be about 3 to 12 minutes.

(d) Irregular Variation The magnetic declination is found to vary suddenly due to some natural causes, such as earthquakes, volcanic erruptions and so on. This variation is known as 'irregular variation'.

11. Dip of the magnetic needle If a needle is perfectly balanced before magnetisation, it does not remain in the balanced position after it is magnetised. This is due to the magnetic influence of the earth. The needle is found to be inclined towards the pole. This inclination of the needle with the horizontal is known as the 'dip of the magnetic needle'.

It is found that the north end of the needle is deflected downwards in the northern hemisphere and that its south end is deflected downwards in the southern hemisphere. The needle is just horizontal at the equator. To balance the dip of the needle, a rider (brass or silver coil) is provided along with it. The rider is placed over the needle at a suitable position to make it horizontal.

12. Local attraction A magnetic needle indicates the north direction when freely suspended or pivoted. But if the needle comes near some magnetic substances, such as iron ore, steel structures, electric cables conveying current; etc. it is found to be deflected from its true direction, and does not show the actual north. This disturbing influence of magnetic substances is known as 'local attraction'.

To detect the presence of local attraction, the fore and back bearings of a line should be taken. If the difference of the fore and back bearings of the line is exactly 180°, then there is no local attraction.

If the FB and BB of a line do not differ by 180°, then the needle is said to be affected by local attraction, provided there is no instrumental error.

To compensate for the effect of local attraction, the amount of error is found out and is equally distributed between the fore and back bearings of the line.

For example, consider the case when

Observed FB of AB = 60°30′
Observed BB of AB = 240°0′

Calculated BB of AB = 60°30° + 180°0′ = 240°30′
∴ Corrected BB of AB = 1/2 (240°0′ + 240°30′) = 240°15′

Hence, Corrected FB of AB = 240°15′ − 180°0′ = 60°15′

13. Method of application of correction

(a) First Method The interior angles of a traverse are calculated from the observed bearings. Then an angular check is applied. The sum of the interior angles should be equal to $(2n - 4) \times 90°$ (n being the number of sides of the traverse). If it is not so, the total error is equally distributed among all the angles of the traverse.

Then, starting from the unaffected line, the bearings of all the lines may be corrected by using the corrected interior angles. This method is very laborious and is not generally employed.

(b) Second Method In this method, the interior angles are not calculated. From the given table, the unaffected line is first detected. Then, commencing from the unaffected line, the bearings of the other affected lines are corrected by finding the amount of correction at each station.

This is an easy method, and one which is generally employed.

Note: If all the lines of a traverse are found to be affected by local attraction, the line with minimum error is identified. The FB and BB of this line are adjusted by distributing the error equally. Then, starting from this adjusted line, the fore and back bearings of other lines are corrected.

3.3 PRINCIPLE OF COMPASS SURVEYING

The principle of compass surveying is traversing, which involves a series of connected lines. The magnetic bearings of the lines are measured by prismatic compass and the distances of the lines are measured by chain. Such survey does not require the formation of a network of triangles.

Interior details are located by taking offsets from the main survey lines. Sometimes subsidiary lines may be taken for locating these details.

Compass surveying is recommended when:

1. A large area to be surveyed
2. The course of a river or coast line is to be surveyed and
3. The area is crowded with many details and triangulation is not possible

Compass surveying is not recommended for areas where local attraction is suspected due to the presence of magnetic substances like steel structures, iron ore deposits, electric cables conveying current, and so on.

3.4 TRAVERSING

As already stated in the last section, surveying which involves a series of connected lines is known as 'traversing.' The sides of the traverse are known as 'traverse legs'.

In traversing, the lengths of the lines are measured by chain and the directions are fixed by compass or theodolite or by forming angles with chain and tape.

A traverse may be of two types—closed and open.

1. Closed traverse When a series of connected lines forms a closed circuit, i.e. when the finishing point coincides with the starting point of a survey, it is called a 'closed traverse'. Here ABCDEA represents a closed traverse (Fig. 3.7). Closed traverse is suitable for the survey of boundaries of ponds, forests, estates, etc.

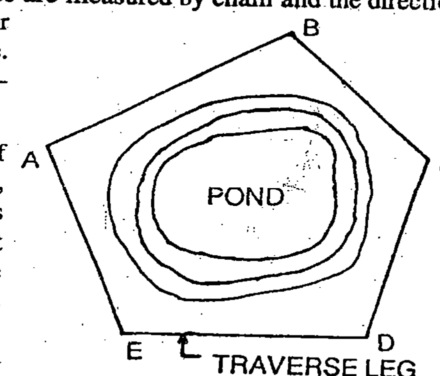


Fig. 3.7 Closed Traverse

2. Open traverse When a sequence of connected lines extends along a general direction and does not return to the starting point, it is known as 'open traverse' or 'unclosed traverse'. Here, ABCDE represents an open traverse (Fig. 3.8).

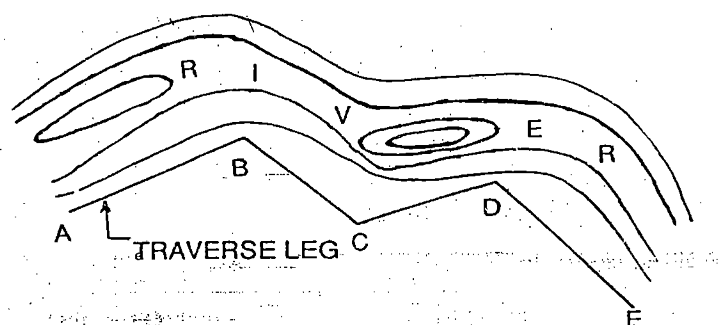


Fig. 3.8 Open Traverse

Open traverse is suitable for the survey of roads, rivers, coast lines, etc.

3.5 METHODS OF TRAVERSING

Traverse survey may be conducted by the following methods:

1. Chain traversing (by chain angle)
2. Compass traversing (by free needle)
3. Theodolite traversing (by fast needle) and
4. Plane table traversing (by plane table)

1. Chain traversing Chain traversing is mainly conducted when it is not possible to adopt triangulation. In this method, the angles between adjacent sides are fixed by chain angles. The entire survey is conducted by chain and tape only and no angular measurements are taken. When it is not possible to form triangles, as, for example, in a pond, chain traversing is conducted, as shown in Fig. 3.9.

The formation of chain angles is explained below.

(a) First Method Suppose a chain angle is to be formed to fix the directions of

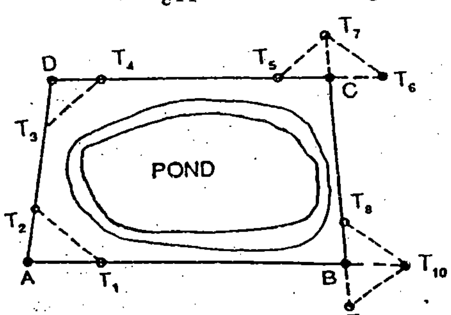


Fig. 3.9 (a) Chain Traversing

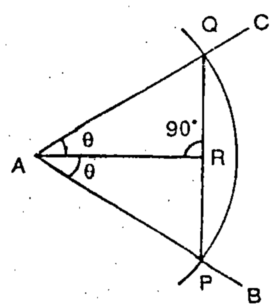


Fig. 3.9(b)

sides AB and AD. Tie stations T_1 and T_2 are fixed on lines AB and AD. The distances AT_1, AT_2 and T_1T_2 are measured. Then the angle $\angle T_1AT_2$ is said to be the chain angle. So, the chain angle is fixed by the tie line T_1T_2. (Fig. 3.9).

(b) Second Method Sometimes the chain angle is fixed by chord. Suppose the angle between the lines AB and AC is to be fixed. Taking A as the centre and a radius equal to one tape length (15 m), an arc intersecting the lines AB and AC at points P and Q, respectively, is drawn. The chord PQ is measured and bisected at R.

Let $\angle PAR = \theta$

Then $\angle BAC = 2\theta$

Here $AP = AQ = 15$ m

In triangle PAR,

$$\sin\theta = \frac{PR}{AP} = \frac{2\,PR}{2\,AP} = \frac{PQ}{30}$$

$$\therefore \quad \theta = \sin^{-1}\frac{PQ}{30}$$

The angle θ can be calculated from the above equation, and the chain angle $\angle BAC$ can be determined accordingly.

2. Compass traversing In this method, the fore and back bearings of the traverse legs are measured by prismatic compass and the sides of the traverse by chain or tape. Then the observed bearings are verified and necessary corrections for local attraction are applied. In this method, closing error may occur when the traverse is plotted. This error is adjusted graphically by using 'Bowditch's rule' (which is described later on).

3. Theodolite traversing In such traversing, the horizontal angles between the traverse legs are measured by theodolite. The lengths of the legs are measured by chain or by employing the stadia method. The magnetic bearing of the starting leg is measured by theodolite. Then the magnetic bearings of the other sides are calculated. The independent coordinates of all the traverse stations are then found out. This method is very accurate.

4. Plane table traversing In this method, a plane table is set at every traverse station in the clockwise or anticlockwise direction, and the circuit is finally closed. During traversing, the sides of the traverse are plotted according to any suitable scale. At the end of the work, any closing error which may occur is adjusted graphically.

3.6 CHECK ON CLOSED TRAVERSE

1. Check on angular measurements

(a) The sum of the measured interior angles should be equal to $(2N - 4) \times 90°$ where N is the number of sides of the traverse.

(b) The sum of the measured exterior angles should be equal to $(2N + 4) \times 90°$.
(c) The algebraic sum of the deflection angles should be equal to 360°.

Right-hand deflection is considered positive and left-hand deflection negative.

2. Check on linear measurement

(a) The lines should be measured once each on two different days (along opposite directions). Both measurements should tally.
(b) Linear measurements should also be taken by the stadia method. The measurements by chaining and by the stadia method should tally.

3.7 CHECK ON OPEN TRAVERSE

In open traverse, the measurements cannot be checked directly. But some field measurements can be taken to check the accuracy of the work. The methods are discussed below.

1. Taking cut-off lines Cut-off lines are taken between some intermediate stations of the open traverse. Suppose ABCDEFG represents an open traverse. Let AD and DG be the cut-off lines. The lengths and magnetic bearings of the cut-off lines are measured accurately. After plotting the traverse, the distances and bearings are noted from the map. These distances and bearings should tally with the actual records obtained from the field (Fig. 3.10).

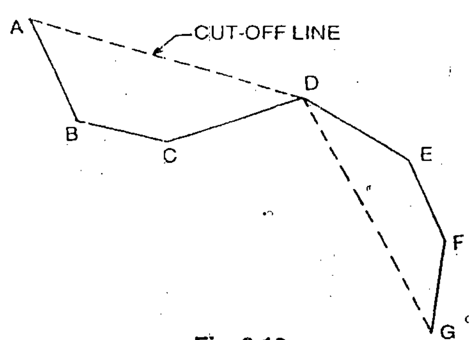


Fig. 3.10

2. Taking an auxiliary point Suppose ABCDEF is an open traverse. A permanent point P is selected on one side of it. The magnetic bearings of this point are taken from the traverse stations A, B, C, D, etc. If the survey is carried out accurately and so is the plotting, all the measured bearings of P when plotted should meet at the point P. The permanent point P is known as the 'auxiliary point' (Fig. 3.11).

3.8 TYPES OF COMPASS

There are two types of compass:

1. The prismatic compass, and

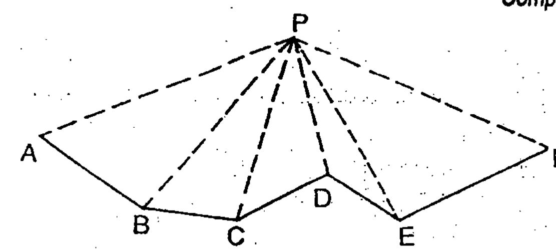


Fig. 3.11

2. The surveyor's compass.

1. The Prismatic compass In this compass, the readings are taken with the help of a prism. The following are the essential parts of this compass:

(a) Compass Box The compass box is a circular metallic box (the metal should be non-magnetic) of diameter 8 to 10 cm. A pivot with a sharp point is provided at the centre of the box.

(b) Magnetic Needle and Graduated Ring The magnetic needle is made of a broad, magnetised iron bar. The bar is pointed at both ends. The magnetic needle is attached to a graduated aluminium ring.

The ring is graduated from 0° to 360° clockwise, and the graduations begin from the south end of the needle. Thus 0° is marked at the south, 90° at the west, 180° at the north and 270° at the east. The degrees are again subdivided into half-degrees. The figures are written upside down. The arrangement of the needle and ring contains an agate cap pivoted on the central pivot point. A rider of brass or silver coil is provided with the needle to counterbalance its dip.

(c) Sight Vane and Prism The sight vane and the reflecting prism are fixed diametrically opposite to the box. The sight vane is hinged with the metal box and consists of a horsehair at the centre. The prism consists of a sighting slit at the top and two small circular holes, one at the bottom of the prism and the other at the side of the observer's eye.

(d) Dark Glasses Two dark glasses are provided with the prism. The red glass is meant for sighting luminous objects at night and the blue glass for reducing the strain on the observer's eye in bright daylight.

(e) Adjustable Mirror A mirror is provided with the sight vane. The mirror can be lowered or raised, and can also be inclined. If any object is too low or too high with respect to the line of sight, the mirror can be adjusted to observe it through reflection.

(f) Brake Pin A brake pin is provided just at the base of the sight vane. If pressed gently, it stops the oscillations of the ring.

(g) Lifting Pin A lifting pin is provided just below the sight vane. When the sight vane is folded, it presses the lifting pin. The lifting pin then lifts the magnetic needle out of the pivot point to prevent damage to the pivot head.

(h) Glass Cover A glass cover is provided on top of the box to protect the aluminium ring from dust (Fig. 3.12).

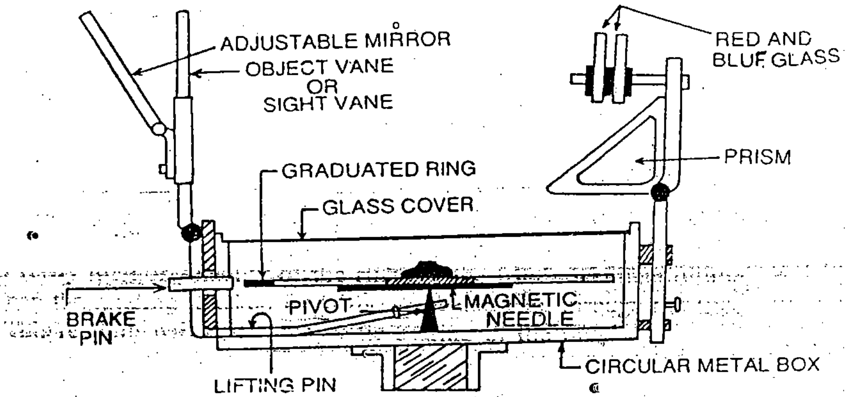


Fig. 3.12

2. The Surveyor's compass The surveyor's compass is similar to the prismatic compass except for the following points.

(a) There is no prism on it. Readings are taken with naked eye.

(b) It consists of an eye-vane (in place of prism) with a fine sight slit.

(c) The graduated aluminium ring is attached to the circular box. It is not fixed to the magnetic needle.

(d) The magnetic needle moves freely over the pivot. The needle shows the reading on the graduated ring.

(e) The ring is graduated from 0° to 90° in four quadrants. 0° is marked at the north and south, and 90° at the east and west. The letters E (east) and W (west) are interchanged from their true positions. The figures are written the right way up.

(f) No mirror is attached to the object vane.

3.9 TEMPORARY ADJUSTMENT OF PRISMATIC COMPASS (FIELD PROCEDURE OF OBSERVING BEARING)

The following procedure should be adopted while measuring the bearing by prismatic compass.

1. Fixing the compass with tripod stand The tripod stand is placed at the required station with its legs well apart. Then the prismatic compass is held by the left hand and placed over the threaded top of the stand. After this, the compass box is turned clockwise by the right hand. Thus the threaded base of the compass box is fixed with the threaded top of the stand.

2. Centering Normally, the compass is centred by dropping a piece of stone from the bottom of the compass box. Centring may also be done with the aid of a plumb bob held centrally below the compass box.

3. Levelling Levelling is done with the help of a ball-and-socket arrangement provided on top of the tripod stand. This arrangement is loosened and the box is placed in such a way that the graduated ring rotates freely without touching either the bottom of the box or the glass cover on top.

4. Adjustment of prism The prism is moved up and down till the figures on the graduated ring are seen sharp and clear.

5. Observation of bearing After centring and levelling the compass box over the station, the ranging rod at the required station is bisected perfectly by sighting through the slit of the prism and horsehair at the sight vane.

At this time the graduated ring may rotate rapidly. The brake pin is pressed very gently to stop this rotation. When the ring comes to rest, the box is struck very lightly to verify the horizontality of the ring and the frictional effect on the pivot point. Then the reading is taken from the graduated ring through the hole in the prism. This reading will be the magnetic bearing of the line.

3.10 PROBLEMS ON WHOLE CIRCLE BEARING AND QUADRANTAL BEARING

Problem 1 Convert the following WCBs to QBs.

(a) WCB of AB = 45°30′
(b) WCB of BC = 125°45′
(c) WCB of CD = 222°15′
(d) WCB of DE = 320°30′

Solution

(a) QB of AB = N 45°30′ E
(b) QB of BC = 180°0′ − 125°45′ = S54°15′ E
(c) QB of CD = 222°15′ − 180°0′ = S42°15′ W
(d) QB of DE = 360°0′ − 320°30′ = N39°30′ W

Problem 2 Convert the following QBs to WCB

(a) QB of AB = S 36°30′ W
(b) QB of BC = S 43°30′ E
(c) QB of CD = N 26°45′ E
(d) QB of DE = N 40°15′ W

Solution

(a) WCB of AB = 180°0′ + 36°30′ = 216°30′
(b) WCB of BC = 180°0′ − 43°30′ = 136°30′
(c) WCB of CD = given QB = 26°45′
(d) WCB of DE = 360°0′ − 40°15′ = 319°45′

3.11 PROBLEMS ON FORE AND BACK BEARINGS

Problem 1 The FBs of the following lines are given. Find the BBs.

(a) FB of AB = 310°30′

(b) FB of BC = 145°15′
(c) FB of CD = 210°30′
(d) FB of DE = 60°45′

Solution
(a) BB of AB = 310°30′ – 180°0′ = 130°30′
(b) BB of BC = 145°15′ + 180°0′ = 325°15′
(c) BB of CD = 210°30′ – 180°0′ = 30°30′
(d) BB of DE = 60°45′ + 180°0′ = 240°45′

Problem 2 FBs of the following lines are given. Find the BBs.
(a) FB of AB = S 30°30′ E
(b) FB of BC = N 40°30′ W
(c) FB of CD = S 60°15′ W
(d) FB of DE = N 45°30′ E

Solution
(a) BB of AB = N 30°30′ W
(b) BB of BC = S 40°30′ E
(c) BB of CD = N 60°15′ E
(d) BB of DE = S 45°30′ W

Problem 3 BBs of the following lines are given. Find the FBs.
(a) BB of AB = 40°30′
(b) BB of BC = 310°45′
(c) BB of CD = 145°45′
(d) BB of DE = 215°30′

Solution
(a) FB of AB = 40°30′ + 180° 0′ = 220°30′
(b) FB of BC = 310°45′ – 180° 0′ = 130°45′
(c) FB of CD = 145°45′ + 180° 0′ = 325°45′
(d) FB of DE = 215°30′ – 180° 0′ = 35°30′

Problem 4 BBs of the following lines are given. Find the FBs.
(a) BB of AB = N 30°30′ W
(b) BB of BC = S 40°15′ E
(c) BB of CD = N 60°45′ E
(d) BB of DE = S 45°30′ W

Solution
(a) FB of AB = S 30°30′ E
(b) FB of BC = N 40°15′ W
(c) FB of CD = S 60°45′ W
(d) FB of DE = N 45°30′ E

3.12 PROBLEMS ON MAGNETIC DECLINATION

Remember the following:

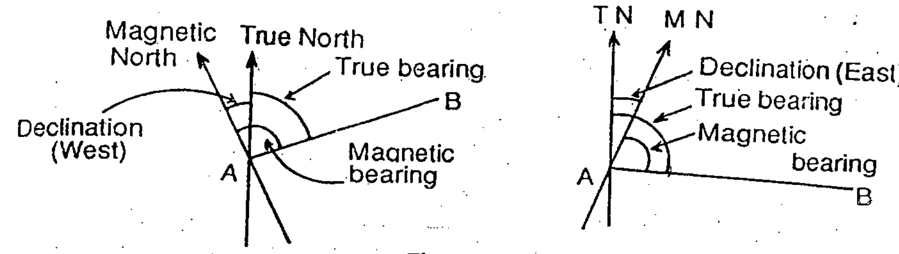


Fig. P-3.1

Determination of true bearing and magnetic bearing:

(a) True bearing = magnetic bearing ± declination

Note [use the positive sign when declination east, and the negative sign when declination west.]

(b) Magnetic bearing = true bearing ± declination

Note [Use the positive sign when declination west, and the negative sign when declination east.]

Problem 1 (a) The magnetic bearing of a line AB is 135°30′. What will be the true bearing, if the declination is 5°15′ W.
(b) The true bearing of a line CD is 210°45′. What will be its magnetic bearing, if the declination is 8°15′ W.

Solution
(a) True bearing of AB = magnetic bearing – declination
= 135°30′ – 5°15′ = 130°15′
(b) Magnetic bearing = true bearing + declination
= 210°45′ + 8°15′ = 219°0′

Problem 2 The magnetic bearing of a line CD is S 30°15′ W. Find its true bearing, if the declination is 10°15′ E.

Solution First convert the RB to WCB, and then follow the usual procedure to find the true bearing in WCB. Finally, convert the true bearing to RB.

RB of CD = S 30°15′ W
WCB of CD = 180°0′ + 30°15′ = 210°15′

Now TB = MB + declination (east)
= 210°15′ + 10°15′ = 220°30′

Required true bearing = 220°30′ – 180° = S 40°30′ W

Problem 3 On an old map a line was drawn to a magnetic bearing of 320°30′, when the declination was 3°30′ W. Find the present bearing of the line, if the declination is 4°15′ E.

Solution

True bearing = magnetic bearing – declination (west)

= 320°30′ – 3°30′ = 317°0′

The true bearing of a line is constant.

So, the present true bearing of the line is also 317° 0′

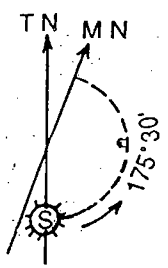
Magnetic bearing = true bearing – declination (east)

= 317°0′ – 4°15′ = 312°45′

Problem 4 (a) The magnetic bearing of the sun at noon is 175°30′ from a station. Find the magnetic declination at that station.

(b) The magnetic bearing of the sun at noon is 5°30′ from other station. Find the magnetic declination at that station.

Solution (a) The sun is exactly on the true meridian at noon. Since the magnetic bearing of the sun is 175°30′, it is towards the south pole of true meridian. Draw a true meridian and the sun place towards the south pole. Then set out an angle of 175°30′ anticlockwise from the sun to get the magnetic meridian.



Magnetic declination = 180°0′–175°30′=4°30′ E

Fig. P-3.2

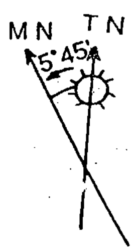


Fig. P-3.3

Thus Magnetic declination = 180°0′ – 175°30′ = 4°30′ E

(b) The sun is exactly on the true meridian at noon. Since the magnetic bearing of the sun is 5°45′, it is towards the north pole of true meridian. Draw the sun towards the north pole of the true meridian. After that, set out an angle of 5°45′ anticlockwise from the sun to get the magnetic meridian.

Thus Magnetic declination = 5°45′ W

3.13 PROBLEMS ON INCLUDED ANGLE

Problem 1 The bearings of the lines OA, OB, OC, OD are 30°30′, 140°15′, 220°45′ and 310°30′, respectively. Find the angles ∠AOB, ∠BOC and ∠COD.

Solution

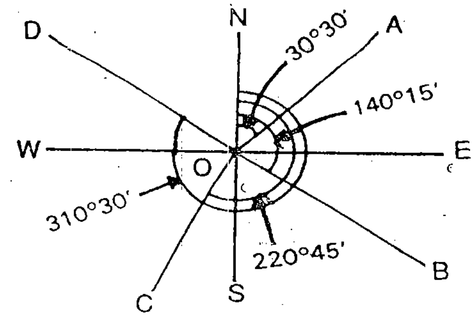


Fig. P.3.4

∠AOB = Bearing of OB – bearing of OA
= 140°15′ – 30°30′ = 109°45′

∠BOC = Bearing of OC – bearing of OB
= 220°45′ – 140°15′ = 80°30′

∠COD = Bearing of OD – bearing of OC
= 310°30′ – 220°45′ = 89°45′

Problem 2 The fore bearings of the lines AB, BC, CD and DE, are 45°30′ 120°15′, 200°30′ and 280°45′, respectively. Find angles ∠B, ∠C and ∠D.

Solution

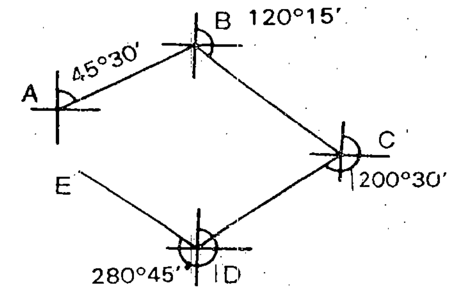


Fig. P-3.5

Interior ∠B = BB of AB – FB of BC
= (45°30′ + 180°0′) – 120°15′
= 225°30′ – 120°15′ = 105°15′

Interior ∠C = BB of BC – FB of CD
= (120°15′ + 180°0′) – 200°30′
= 300°15′ – 200°30′ = 99°45′

Exterior ∠D = FB of DE – BB of CD
= 280°45′ – (200°30′ – 180°0′)
= 280°45′ – 20°30′ = 260°15′

Interior ∠D = 360°0′ – 260°15′ = 99°45′

Problem 3 A traverse is done by three stations A, B and C in clockwise order in the form of an equilateral triangle. If the bearing of AB is 80° 30′, find the bearings of the other sides.

Solution

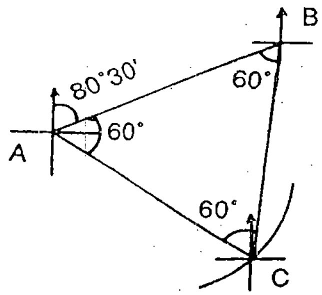


Fig. P-3.6

FB of AB = 80° 30′
FB of BC = BB of AB – ∠B
= (80°30′ + 180°0′) – 60°0′
= 260°30′ – 60°0′ = 200°30′

FB of CA = BB of BC + exterior ∠C
= (200°30′ – 180°0′) + (360°0′ – 60°0′)
= 20°30′ + 300°0′ = 320°30′

FB of AB = BB of CA – ∠A
= (320°30′ – 180°0′) – 60°0′
= 140°30′ – 60°0′ = 80°30′ (checked)

Problem 4 A traverse ABCDA is made in the form of a square taking in clockwise order. If the bearing of AB is 120°30′, find the bearing of the other sides.

Solution

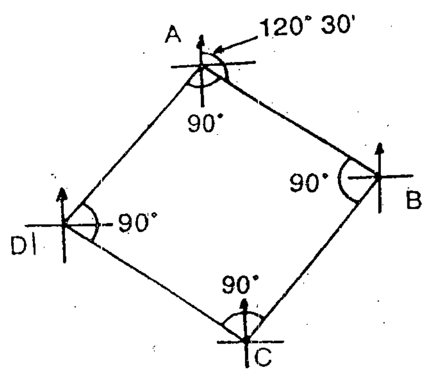


Fig. P-3.7

FB of AB = 120° 30′
FB of BC = BB of AB – ∠B
= (120°30′ + 180°0′) – 90°0′
= 300°30′ – 90°0′ = 210°30′

FB of CD = BB of BC + exterior ∠C
= (210°30′ – 180°0′) + (360° – 90°)
= 30°30′ + 270°0′ = 300°30′

FB of DA = BB of CD – ∠D
= (300°30′ – 180°0′) – 90°0′
= 120°30′ – 90°0′ = 30°30′

FB of AB = BB of DA – ∠A
= (30°30′ + 180°0′) – 90°0′
= 210°30′ – 90°0′ = 120°30′ (checked)

Problem 5 A closed traverse is conducted with five stations A, B, C, D and E taken in anticlockwise order, in the form of a regular pentagon. If the FB of AB is 30°0′, find the FBs of the other sides.

Solution

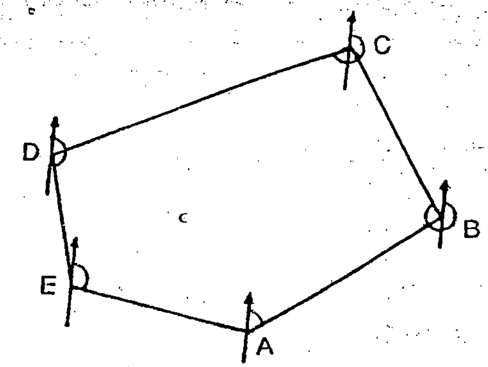


Fig. P-3.8

$$\text{Interior angle of pentagon} = \frac{(2N-4)\times 90°}{5} = \frac{540°}{5} = 108°$$

FB of AB = 30°0′
FB of BC = BB of AB + ∠B
= (30°0′ + 180°0′) + 108°0′
= 210°0′ + 108°0′ = 318°0′

FB of CD = BB of BC + ∠C
= (318°0′ – 180°0′) + 108°0′
= 138°0′ + 108°0′ = 246°0′

FB of DE = BB of CD + ∠D
= (246°0′ + 180°0′) + 108°0′
= 66°0′ + 108°0′ = 174°0′

FB of EA = BB of DE – exterior ∠E
= (174°0′ + 180°0′) – (360°0′ – 108°0′)
= 354°0′ – 252° = 102°0′

FB of AB = BB of EA − exterior ∠A
= (102°0′ + 180°0′) − (360°0′ − 108°0′)
= 282°0′ − 252°0′ = 30°0′ (checked)

Problem 6 The following are the fore and back bearings of the sides of a closed traverse:

Side	FB	BB
AB	150°15′	330°15′
BC	20°30′	200°30′
CD	295°45′	115°45′
DE	218°0′	38°0′
EA	120°30′	300°30′

Calculate the interior angles of the traverse.

Solution

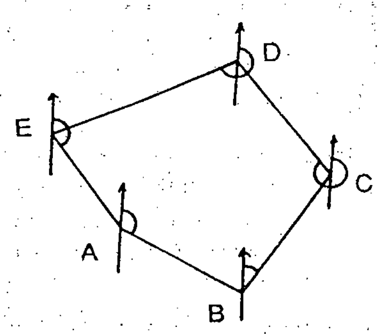


Fig. P-3.9

Exterior ∠A = BB of EA − FB of AB
= 300°30′ − 150°15′ = 150°15′

(a) Interior ∠A = 360°0′ − 150°15′ = 209°45′
Exterior ∠B = BB of AB − FB of BC
= 330°15′ − 20°30′ = 309°45′

(b) Interior ∠B = 360°0′ − 309°45′ = 50°15′

(c) Interior ∠C = FB of CD − BB of BC
= 295°45′ − 200°30′ = 95°15′

(d) Interior ∠D = FB of DE − BB of CD
= 218°0′ − 115°45′ = 102°15′

(e) Interior ∠E = FB of EA − BB of DE
= 120°30′ − 38°0′ = 82°30′

Check The sum of the interior angles should be equal to $(2N - 4) \times 90°$. In this case,

$$(2N - 4) \times 90 = 540° \qquad (N = 5)$$

Sum of calculated interior angles = ∠A + ∠B + ∠C + ∠D + ∠E
= 209°45′ + 50°15′ + 95°15′ + 102°15′ + 82°30′
= 540° (checked and found correct)

Problem 7 The following are the bearings of a closed traverse:

Side	FB	BB
AB	N 45°30′ E	S 45°30′ W
BC	S 60°0′ E	N 60°0′ W
CD	S 10°30′ W	N 10°30′ E
DA	N 75°45′ W	S 75°45′ E

Calculate the interior angles of the traverse.

Solution

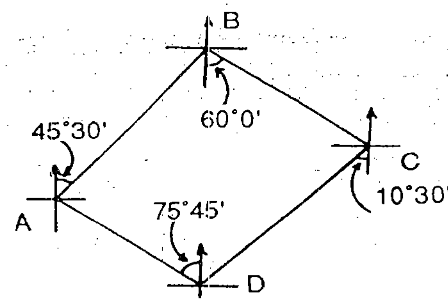


Fig. P-3.10

Interior ∠A = 180° − (FB of AB + BB of DA)
= 180° − (45°30′ + 75°45′) = 180°0′ − 121°15′ = 58°45′
Interior ∠B = BB of AB + FB of BC
= 45°30′ + 60°0′ = 105°30′
Interior ∠C = 180°0′ − (BB of BC + FB of CD)
= 180°0′ − (60°0′ + 10°30′) = 180°0′ − 70°30′ = 109°30′
Interior ∠D = BB of CD + FB of DA
= 10°30′ + 75°45′ = 86°15′

Check The sum of the interior angles should be

$$(2N - 4) \times 90° = 360°$$

Now ∠A + ∠B + ∠C + ∠D = 58°45′ + 105°30′ + 109°30′ + 86°15′ = 360°
(checked and found correct)

Problem 8 The following are the bearings observed in traversing, with a compass, an area where local attraction was suspected. Calculate the interior angles of the traverse and correct them if necessary.

Line	FB	BB
AB	150°0′	330°0′
BC	230°30′	48°0′
CD	306°15′	127°45′
DE	298°00′	120°00′
EA	49°30′	229°30′

Solution

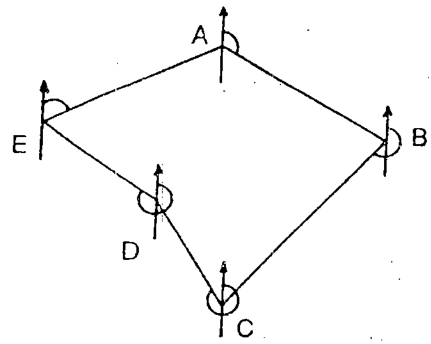


Fig. P-3.11

(a) Interior ∠A = BB of EA – FB of AB
= 229°30′ – 150°0′ = 79°30′

(b) Interior ∠B = BB of AB – FB of BC
= 330°0′ – 230°30′ = 99°30′

Exterior ∠C = FB of CD – BB of BC
= 306°15′ – 48°0′ = 258°15′

(c) Interior ∠C = 360°0′ – 258°15′ = 101°45′

Exterior ∠D = FB of DE – BB of CD
= 298°00′ – 127°45′ = 170°15′

(d) Interior ∠D = 360°0′ – 170°15′ = 189°45′

(e) Interior ∠E = BB of DE – FB of EA
= 120°0′ – 49°30′ = 70°30′

Check Sum of interior angles = ∠A + ∠B + ∠C + ∠D + ∠E
= 541°0′

But, the sum of angles should be $(2N-4)\times 90° = 540°0'$

Here, Error = 541° – 540° = + 1°

$$\text{Correction per angle} = -\frac{60'}{5} = -12'$$

The error should equally distributed among all the angles.

Angle	Calculated value	Correction	Corrected value
∠A	79°30′	– 12′	79°18′
∠B	99°30′	– 12′	99°18′
∠C	101°45′	– 12′	101°33′
∠D	189°45′	– 12′	189°33′
∠E	70°30′	– 12′	70°18′
	Total = 541°0′		540°00′

3.14 PROBLEMS ON LOCAL ATTRACTION

Problem 1 The following are the observed bearings of the lines of a traverse ABCDEA with a compass in a place where local attraction was suspected.

Line	FB	BB
AB	191°45′	13°0′
BC	39°30′	222°30′
CD	22°15′	200°30′
DE	242°45′	62°45′
EA	330°15′	147°45′

Find the correct bearings of the lines. (WBSC 1969)

Solution First method—By calculating interior angles

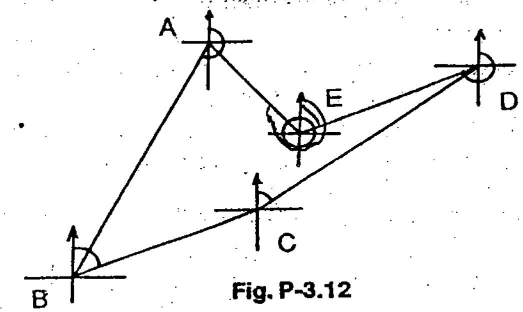


Fig. P-3.12

(a) Calculation of interior angle

Interior ∠A = FB of AB – BB of EA = 191°45′ – 147°45′ = 44°00′
Interior ∠B = FB of BC – BB of AB = 39°30′ – 13°00′ = 26°30′
Exterior ∠C = BB of BC – FB of CD = 222°30′ – 22°15′ = 200°15′
Interior ∠C = 360°00′ – 200°15′ = 159°45′
Interior ∠D = FB of DE – BB of CD
= 242°45′ – 200°30′ = 42°15′
Interior ∠E = FB of EA – BB of DE
= 330°15′ – 62°45′ = 267°30′

Sum of interior angles = 44°00′ + 26°30′ + 159°45′ + 42°15′ + 267°30′
= 540°00′

which is equal to $(2N-4)\times 90° = 540°00'$

So, the calculated angles are correct.

(b) Calculation of corrected bearing

The line DE is free from local attraction. So,

FB of DE = 242°45′ (correct)
and FB of EA = 330°15′ (correct)

FB of AB = BB of EA + ∠A
= (330°15′ – 180°0′) + 44°00′
= 150°15′ + 44°00′ = 194°15′
FB of BC = BB of AB + ∠B
= (194°15′ – 180°0′) + 26°30′
= 14°15′ + 26°30′ = 40°45′
FB of CD = BB of BC – exterior ∠C
= (40°45′ + 180°00′) – 200°15′
= 220°45′ – 200°15′ = 20°30′
FB of DE = BB of CD + ∠D
= (20°30′ + 180°0′) + 42°15′
= 200°30′ + 42°15′
= 242°45′ (checked)

The result is tabulated as follows:

Line	Corrected	
	FB	BB
AB	194°15′	14°15′
BC	40°45′	220°45′
CD	20° 30′	200°30′
DE	242°45′	62°45′
EA	330°15′	150°15′

Second method—Directly applying correction

Procedure (a) On verifying the observed bearing it is found that the FB and BB of line DE differ by exactly 180°. So, the stations D and E are free from local attraction and the observed FB and BB of DE are correct.

(b) The observed FB of EA is also correct.

(c) The actual BB of EA should be

330°15′ – 180°0′ = 150°15′

But the observed bearing is 147°45′.
So, a correction of (150°15′ – 147°45′) = + 2°30′ should be applied at A.

(d) Correct FB of AB = 191°45′ + 2°30′ = 194°15′
Therefore, the actual correct BB of AB should be

194°15′ – 180°00′ = 14°15′

But Observed bearing = 13°0′

So, a correction of (14°15′ – 13°0′) = + 1°15′ should be applied at B.

(e) Correct FB of BC = 39° 30′ + 1°15′ = 40°45′
∴ Correct BB of BC should be = 40°45′ + 180°0′ = 220°45′
But Observed bearing of BC = 222°30′
So, a correction of

(220°45′ – 222°30′) = – 1°45′ should be applied at C.

(f) Correct FB of CD = 22°15′ – 1°45′ = 20°30′
Therefore, the BB of CD should be

20°30′ + 180°0′ = 200°30′

which tallies with the observed BB of CD.

So, D is free from local attraction, which also tallies with the remark made at the beginning.

The result is tabulated as follows:

Table for Correction

Line	Observed		Correction	Correct		Remarks
	FB	BB		FB	BB	
AB	191°45′	13°00′	+ 2°30′ at A	194°15′	14°15′	
BC	39°30′	222°30′	+ 1°15′ at B	40°45′	220°45′	
CD	22°15′	200°30′	– 1°45′ at C	20°30′	200°30′	
DE	242°45′	62°45′	0° at D	242°45′	62°45′	Station D is free from local attraction
EA	330°15′	147°45′	0° at E	330°15′	150°15′	Station E is also free from local attraction

Problem 2 The following bearings were observed in traversing, with a compass, an area where local attraction was suspected. Find the amounts of local attraction at different stations, the correct bearings of lines and the included angles. Also draw a sketch of the plot if AB = 100 m, BC = 100 m and CD = 50 m and show in it all the included angles. (WBSC 1989).

Line	FB	BB
AB	68°15′	248°15′
BC	148°45′	326°15′
CD	224°30′	46°00′
DE	217°15′	38°15′
EA	327°45′	147°45′

Solution

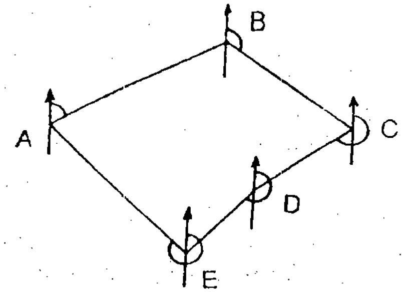


Fig. P-3.13

(a) *Calculation of Included angles*

Included ∠A = BB of EA – FB of AB
= 147°45′ – 68°15′ = 79°30′
Included ∠B = BB of AB – FB of BC
= 248°15′ – 148°45′ = 99°30′
Included ∠C = BB of BC – FB of CD
= 326°15′ – 224°30′ = 101°45′
Exterior ∠D = FB of DE – BB of CD
= 217°15′ – 46°00′ = 171°15′
Included ∠D = 360°00′ – 171°15′ = 188°45′
Exterior ∠E = FB of EA – BB of DE
= 327°45′ – 38°15′ = 289°30′
Included ∠E = 360°0′ – 289°30′ = 70°30′

Check

$(2N - 4) \times 90 = (2 \times 5 - 4) \times 90° = 540°00'$
$\angle A + \angle B + \angle C + \angle D + \angle E = 540°00'$

(b) *Calculation of corrected bearings*

Procedure (i) On verifying the observed bearings, it is found that the difference of FB and BB of the line AB is exactly 180°. So, A and B are free from local attraction.

(ii) The observed FB of BC is also correct

Therefore, the actual BB of BC should be

(148°45′ + 180°0′) = 328°45′

But the observed bearing is 326°15′.
So, a correction of (328°45′ – 326°15′) = + 2°30′ should be applied at C.

(iii) Correct FB of CD = 224°30′ + 2°30′ = 227°0′
Therefore, the actual BB of CD should be

227°0′ – 180°0′ = 47°00′

But observed bearing of CD = 46°0′
So, a correction of (47°0′ – 46°0′) = + 1°0′ should be applied at D.

(iv) Correct FB of DE = 217°15′ + 1°0′ = 218°15′
Therefore, the correct BB of DE should be

218°15′ – 180°0′ = 38°15′

which is equal to the observed BB of DE. So, station E is also free from local attraction.

(v) Since stations A and E are both free from local attraction, the FB and BB of EA are correct.

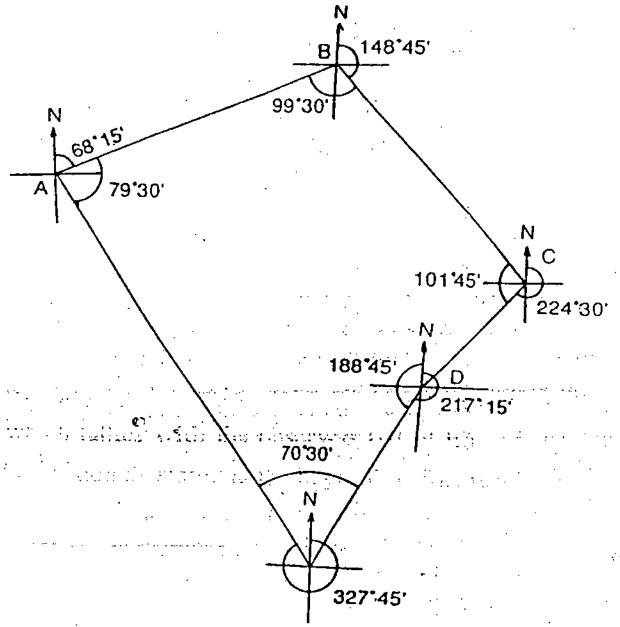
The result is tabulated as follows:

Line	Observed		Correction	Correct		Remarks
	FB	BB		FB	BB	
AB	68°15′	248°15′	0° at A	68°15′	248°15′	Station A is free from local attraction
BC	148°46′	326°15′	0° at B	148°45′	328°45′	Station B is free from local attraction
CD	224°30′	46°00′	+2°30′ at C	227°00′	47°00′	
DE	217°15′	38°15′	+1°0′ at D	218°15′	38°15′	
EA	327°45′	147°45′	0° at E	327°45′	147°45′	Station E is free from local attraction

(c) *Sketch of the plot*

AB = 100 m, BC = 100 m, CD = 50 m
Scale assumed 1 cm = 20 m

The north line is assumed vertical. The bearings of all the lines are set out by protractor. The given distances are plotted to the assumed scale, as shown in Fig. P-3.14.



SCALE—R F = $\frac{1}{2000}$

Fig. P-3.14

Problem 3 Followings are the bearings observed while traversing with a compass, an area where local attraction was suspected. Find the correct bearings of the lines and also the true bearings, if the magnetic declination is 10°W.

Line	FB	BB
AB	59°00′	239°00′
BC	139°30′	317°00′
CD	215°15′	36°30′
DE	208°00′	29°00′
EA	318°30′	138°45′

Solution

Procedure (*a*) On verifying the observed bearings, it is found that the FB and BB of AB differ by exactly 180°. So stations A and B are free from local attraction. Hence the observed FB and BB of AB are correct.

(*b*) The observed FB of BC is also correct.

The actual BB of BC should be 139°30′ + 180°0′ = 319°30′

But the observed BB is 317°0′.

So, a correction of (319°30′ – 317°0′) = + 2°30′) should be applied at C.

(*c*) Correct FB of CD = 215°15′ + 2°30′ = 217°45′

Therefore, the actual BB should be 217°45′ – 180°0′ = 37°45′

But the observed BB is 36°30′.

So, a correction of (37°45′ – 36°30′) = + 1°15′ should be applied at D.

(*d*) Correct FB of DE = 208°00′ + 1°15′ = 209°15′

The correct BB should be (209°15′ – 180°0′) = 29°15′

But the observed BB is 29°0′.

So, a correction of (29°15′ – 29°0′) = + 0°15′ should be applied at E.

(*e*) Correct FB of EA = 318°30′ + 0°15′ = 318°45′

The actual BB of EA should be

318°45′ – 180°0′ = 138°45′ which tallies with the observed BB of EA.

So, station A is free from local attraction as stated at the beginning. The result is tabulated as follows:

Line	Observed		Correction	Correct		Remarks
	FB	BB		FB	BB	
AB	59°00′	239°00′	0° at A	59°00′	239°00′	Station A is free from local attraction
BC	139°30′	317°00′	0° at B	139°30′	319°30′	Station B is free from local attraction.
CD	215°15′	36°30′	+2°30′ at C	217°45′	37°45′	
DE	208°00′	29°00′	+1°15′ at D	209°15′	29°15′	
EA	318°30′	138°45′	+0°15′ at E	318°45′	138°45′	

Table for True Bearing

Line	Correct		Declination	True		Remark
	FB	BB	10° W	FB	BB	
AB	59°00′	239°00′	– 10°	49°00′	229°00′	True bearing is obtained by deducting declination from magnetic bearingas declination is west.
BC	139°30′	319°30′	– 10°	129°30′	309°30′	
CD	217°45′	37°45′	– 10°	207°45′	27°45′	
DE	209°15′	29°15′	– 10°	199°15′	19°15′	
EA	318°45′	138°45′	– 10°	308°45′	128°45′	

Problem 4 The following bearings were observed at a place where local attraction was suspected. Find the corrected bearings of the lines.

Line	FB	BB
AB	S 45°30′ E	N 45°30′ W
BC	S 60°0′ E	N 60°40′ W
CD	S 5°30′ E	N 3°20′ W
DA	N 83°30′ W	S 85°0′ E

Solution

Procedure (*a*) The FB and BB of AB are numerically equal but their quadrants are just opposite. So, stations A and B are free from local attraction. Hence, the given FB and BB of AB are correct.

(*b*) FB of BC = S 60°0′ E (correct)

Therefore, the actual BB of BC should be N 60°0′ W

But the observed BB is N 60°40′ W.

So, a correction of (60°0′ – 60°40′) = – 0°40′ is to be applied at C.

(*c*) Correct FB of CD = S 5°30′ E – 0°40′ = S 4°50′ E

Therefore, the correct BB of CD should be N 4°50′ W

But the observed BB of CD is N 3°20′ W.

So, a correction of (4°50′ – 3°20′) = + 1°30′ is to be applied at D.

(*d*) Correct FB of DA = N 83°30′ W + 1°30′ = N 85°0′ W

The correct BB of DA should be S 85° 0′ E, which tallies with the observed BB of DA.

So, A is free from local attraction, is stated at the beginning.

The result is tabulated as follows.

Line	Observed		Correction	Corrected		Remarks
	FB	BB		FB	BB	
AB	S 45°30′ E	N 45°30′ W	0° at A	S45°30′ E	N 45°30′ W	
BC	S 60°0′ E	N 60°40′ W	0° at B	S 60°0′ E	N 60°0′ W	
CD	S 5°30′ E	N 3°20′ W	– 0°40′ at C	S 4°50′ E	N 4°50′ W	
DA	N 83°30′ W	S 85°0′ E	+ 1°30′ at D	N 85°0′ W	S 85°0′ E	

Problem 5 The following bearings were observed where local attraction was suspected. Calculate the actual bearings.

Line	FB	BB
AB	S40°30′ W	N 41°15′ E
BC	S 80°45′ W	N 79°30′ E
CD	N 19°30′ E	S 20°00′ W
DA	S 80°00′ E	N 80°00′ W

Solution

Procedure (a) The FB and BB of DA are numerically equal but their quadrants are exactly opposite. So, stations D and A are free from local attraction. Hence, the observed FB and BB of DA are correct.

(b) FB of AB = S 40°30′ W (correct)

Therefore, the actual BB of AB should be N 40°30′ E

But the observed BB of AB is N 41°15′ E.

So, a correction of (40°30′ – 41°15′) = – 0°45′ is to be applied at B.

(*c*) Correct FB of BC = S 80°45′ W – 0°45′ = S 80°0′ W

Therefore, the actual BB should be N 80°0′ E.

But the observed BB of BC is N 79°30′ E.

So, a correction of (80°0′ – 79°30′) = + 0°30′ is to be applied at C.

(*d*) Correct FB of CD = N 19°30′ E + 0°30′ = N 20°0′ E

The actual BB of CD should be S 20°0′ W.

But the observed BB of CD is S 20°0′ W.

So, station D is free from local attraction, which tallies with previous comment.

The result is tabulated as follows:

Line	Observed		Correction	Correct		Remarks
	FB	BB		FB	BB	
AB	S 40°30′ W	N 41°15′ E	0° at A	S 40°30′ W	N 40°30′ E	Station A is free from local attraction.
BC	S 80°45′ W	N 79°30′ E	– 0° 45′ at B	S 80°0′ W	N 80°0′ E	
CD	N 19°30′ E	S 20°0′ W	+ 0°30′ at C	N 20°0′ E	S 20°0′ W	
DA	S 80°0′ E	N 90°0′ W	0° at D	S 80°0′ E	N 80°0′ W	Station D is free from local attraction.

Problem 6 The following are the bearings observed in traversing, with a compass, an area where local attraction was suspected. Find the amount of local attraction at different stations, the correct bearings of the lines, and the included angles. Draw a sketch of the plot assuming AB = 180 m, BC = 120 m, CD = 60 m and show in it all the included angles. (WBSC, 1988)

Line	FB	BB
AB	59°00′	239°00′
BC	139°30′	317°00′
CD	215°15′	36°30′
DE	208°00′	29°00′
EA	318°30′	138°45′

Solution *(a) Calculation of included angles:*

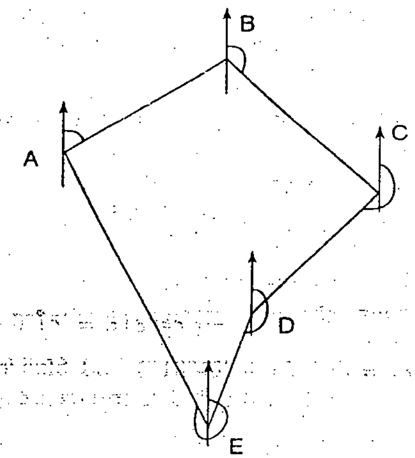


Fig. P-3.15

Included ∠A = BB of EA – FB of AB
= 138°45′ – 59°00′ = 79°45′
Included ∠B = BB of AB – FB of BC
= 239°00′ – 139°30′ = 99°30′
Included ∠C = BB of BC – FB of CD
= 317°00′ – 215°15′ = 101°45′
Exterior ∠D = FB of DE – BB of CD
= 208°00′ – 36°30′ = 171°30′
Included ∠D = 360°00′ – 171°30′ = 188°30′
Exterior ∠E = FB of EA – BB of DE
= 318°30′ – 29°00′ = 289°30′
Included ∠E = 360°00′ – 289°30′ = 70°30′

Check Sum of included angles = $(2N - 4)\ 90° = 540°00′$

Sum of calculated angles = ∠A + ∠B + ∠C + ∠D + ∠E = 540°00′ (correct)

(b) Calculation of corrected bearings:

(i) On verifying the observed bearings it is found that the FB and BB of AB differ by exactly 180°. So, stations A and B are free from local attraction. Hence, the observed FB and BB of AB are correct.

(ii) FB of BC = 139°30′ (correct)

Therefore, the correct BB of BC should be 139°30′ + 180°0′ = 319°30′

But the observed BB of BC is 317°00′.

So, a correction of (319°30′ – 317°00′) = + 2°30′ is to be applied at C.

(iii) Correct FB of CD = 215°15′ + 2°30′ = 217°45′

Therefore, the actual BB of CD should be (217°45′ – 180°0′) = 37°45′

But the observed BB of CD is 36°30′.

So, a correction of (37°45′ – 36°30′) = + 1°15′ should be applied at D.

(iv) Correct FB of DE = 208°00′ + 1°15′ = 209°15′

Therefore, the correct BB of DE should be 209°15′ – 180°0′ = 29°15′

But the observed BB of DE is 29°00′.

So, a correction of (29°15′ – 29°0′) = + 0°15′ is to be applied at E.

(v) Correct FB of EA = 318°30′ + 0°15′ = 318°45′

Therefore, the correct BB of EA should be (318°45′ – 180°0′) = 138°45′ which tallies with the observed bearing. So, station A is free from local attraction, as stated at the beginning.

The result tabulated as follows:

Line	Observed		Correction	Corrected		Remarks
	FB	BB		FB	BB	
AB	59°00′	239°00′	0° at A	59°00′	239°00′	Station A and Station B are free from local attraction
BC	139°30′	317°00′	0° at B	139°30′	319°30′	
CD	215°15′	36°30′	+2°30′ at C	217°45′	37°45′	
DE	208°00′	29°00′	+1°15′ at D	209°15′	29°15′	
EA	318°30′	138°45′	+ 0°15′ at E	318°45′	138°45′	

(c) Sketch of the plot:

AB = 180 m, BC = 120 m, CD = 60 m

Scale assumed, 1 cm = 30 m

The north line is assumed vertical and the bearings are set out by protractor. The distances are plotted to the assumed scale (Fig. P-3.16).

Equipments Required for Compass Survey

The following equipments are required for conducting a compass survey.

1. Prismatic compass with stand = 1 no.
2. Metric chain (20 m) = 1 no.
3. Metallic tape (15 m) = 1 no.
4. Arrows = 10 nos
5. Ranging rods = 3 nos

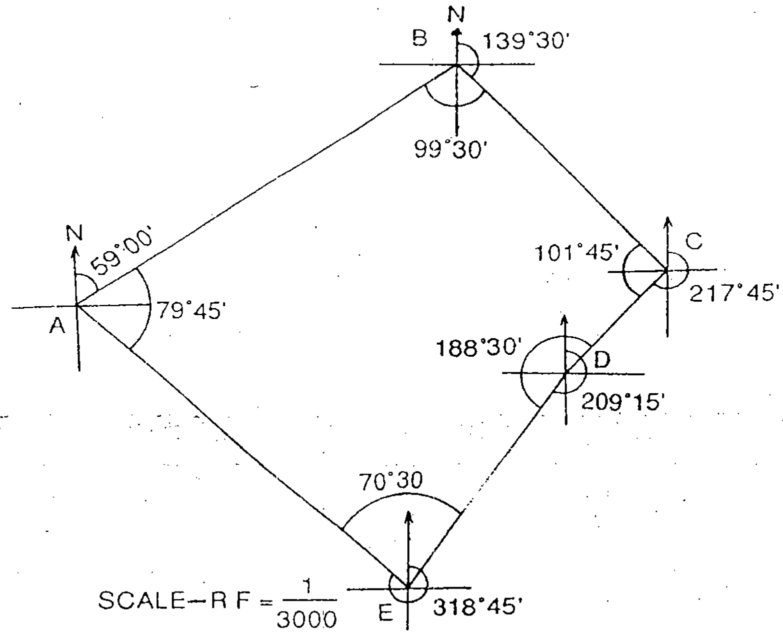


Fig: P-3.16

6. Cross-staff or optical square = 1 no.
7. Plumb-bob = 1 set
8. Tri-square or wooden set square = 1 no.
9. Wooden pegs = 10 nos
10. Mallet or hammer = 1 no.
11. Field book = 1 no.
12. Good pencil = 1 no.
13. Eraser = 1 no.
14. Knife = 1 no.

3.15 FIELD PROCEDURE OF COMPASS TRAVERSING

Prismatic compass traversing is conducted in accordance with the following steps:

(a) Reconnaissance The area to be surveyed is examined thoroughly to select the traverse stations. These stations should be intervisible and should cover the whole area. It should be ensured that there is no magnetic substance near the selected stations. The traverse legs should run along fairly level ground.

(b) Preparation of index sketch After reconnaissance, an index sketch should be prepared showing the skeleton of the traverse.

(c) Marking the station on the ground The traverse stations are marked on the ground by wooden pegs. The pegs should be fixed on the station points in such a way that a height of about 3 cm is always exposed above the ground surface.

Reference sketches should be prepared for all the traverse stations by taking at least two measurements from some permanent points. This precaution is taken so that the stations can be located accurately even if the pegs have been removed by somebody.

(d) Measurement of bearings of traverse legs The traverse stations may be selected clockwise or anticlockwise order. But the direction of the traverse should be indicated in the index sketch. Suppose four traverse stations A, B, C and D are selected to enclose an area (Fig. 3.13). The prismatic compass is centred and levelled at the starting station A. The fore bearing of AB and back bearing of DA are taken from this station. The distance AB is measured and offsets are taken along the line AB and recorded in the field book.

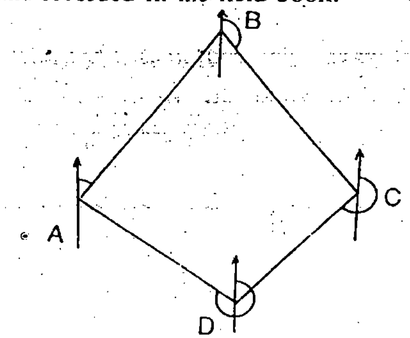


Fig. 3.13

The compass is then shifted and centred over station B. Then the FB of BC and BB of AB are taken. Here, the FB and BB of line AB should differ by exactly 180°. However, the error should not exceed the permissible limit. If it does, the bearings of the lines should be observed again. Now, the line BC is measured and offsets are taken and noted in the field book. Similarly, all the traverse legs are measured and noted in the field book.

After completion of the work, the observations are tabulated and necessary corrections applied to eliminate the effect of local attraction.

3.16 PLOTTING OF COMPASS TRAVERSE

The following are the various methods of plotting compass traverse.

(a) By parallel meridian through each station The starting point A is suitably selected on the paper and a line representing the north line. The bearing of the line AB is plotted by protractor and its length is plotted to any suitable scale.

At station B, the north line is drawn parallel to the north line which was drawn at A. Then the bearing of the line BC is plotted and its length marked according to the previous scale.

Similarly, all the traverse legs are plotted. In case of closed traverse, there may be a closing error which should be adjusted graphically (Fig. 3.14).

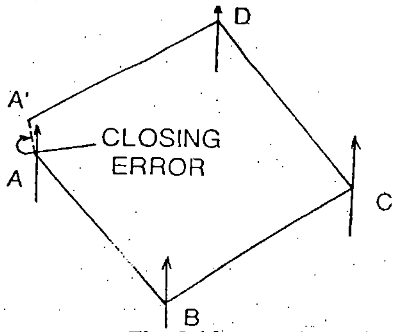


Fig. 3.14

(b) By considering included angles The starting station A is suitably selected on the sheet. A line representing the north line is drawn through station A. The bearing of the line AB is plotted by protractor and the distance AB marked to a suitable scale. At station B the angle ∠B is plotted and the distance BC marked according to the previous scale. Angle ∠C is plotted at station C and the distance CD is marked.

This process is continued until all the lines have been plotted. In this case also, there may be a closing error which has to be adjusted graphically (Fig. 3.15).

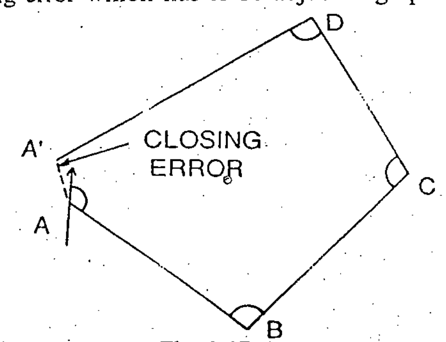


Fig. 3.15

(c) By considering the central meridian A suitable point O is selected at the centre of the drawing sheet. A line representing the magnetic meridian is drawn through this point. Then a protractor is placed at O and all the lines, namely ab, bc, cd and da, are drawn according to their bearings.

Then a starting point A is suitably selected on the sheet. A line AB is drawn parallel to ab, and the length AB is plotted to a suitable scale. Again from B, a line BC is drawn parallel to the line bc and the distance BC is plotted to the previous scale.

The process is continued until all the lines have been drawn. In this case also there may be a closing error is adjusted graphically (Fig. 3.16).

Note: After adjustment of the closing error, the objects are plotted according to the offsets noted in the field book.

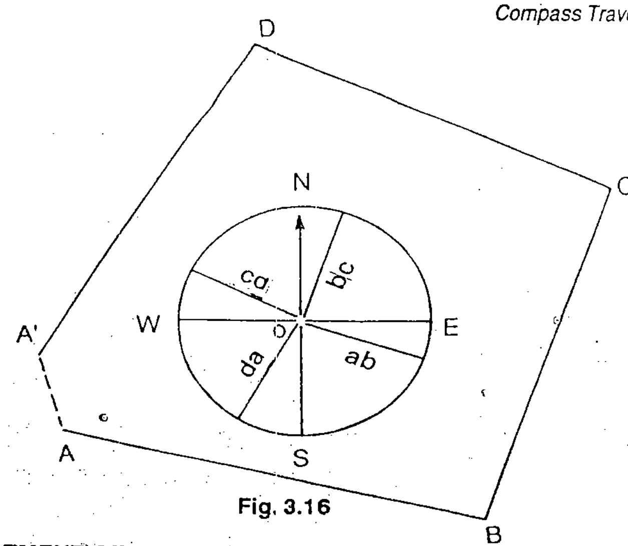


Fig. 3.16

3.17 ADJUSTMENT OF CLOSING ERROR

When a closed traverse is plotted, the finishing and starting points may not coincide. The distance by which the traverse fails to close is said to be the closing error. Such an error may occur due to mistakes made in the measurement of lengths and bearings of the lines, or because of an error in plotting.

If the closing error exceeds a certain permissible limit, the field work should be repeated. But when the error is within the permissible limit, it is adjusted graphically by Bowditch's rule as explained below.

Suppose a traverse $AB_1C_1D_1E_1A_1$ is plotted according to any suitable scale (RF = 1/400) (Fig. 3.17(a)).

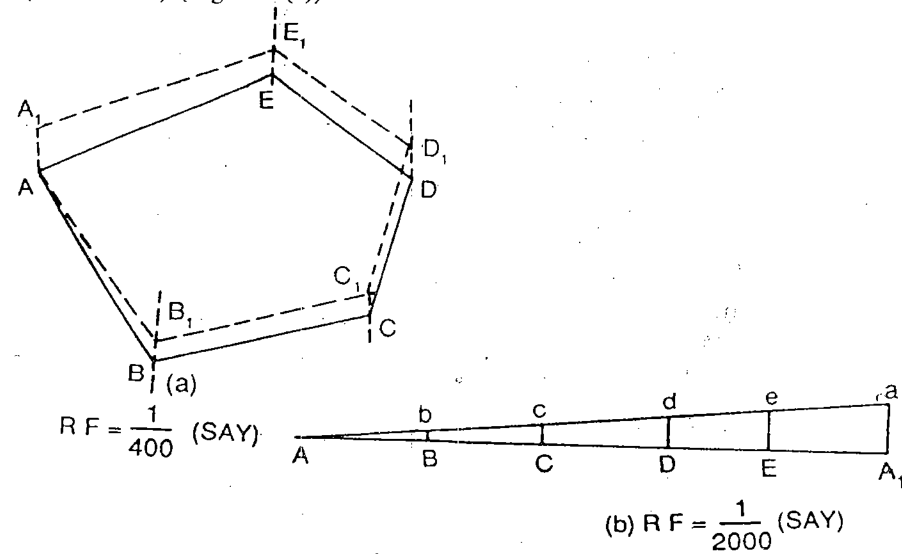


Fig. 3.17 (a) & (b)

In this case, the traverse fails to close by a distance AA_1, which is the closing error.

To adjust this error, a horizontal AA_1 is drawn to represent the perimeter of the traverse to another scale (RF = 1/2,000). On this line, distances AB_1, B_1C_1, C_1D_1, D_1E_1 and E_1A_1 are set off according to the corresponding measured lengths of the traverse legs. A perpendicular A_1a is drawn equal to the amount of closing error, after which the line Aa is drawn. From the points B_1, C_1, D_1 and E_1, the lines B_1b, C_1c, D_1d and E_1e are drawn parallel to A_1 a (Fig. 3.17(b)). These intercepts represent the amount by which the respective stations are to be shifted.

In Fig. 3.17(a), lines are drawn parallel to the closing error through stations B_1, C_1, D_1 and E_1. Then the intercepts B_1b, C_1c, D_1d and E_1e are set off along the parallel lines drawn through the respective stations. In this manner, the adjusted traverse ABCDEA is obtained.

Limits of closing error The angular error of closure should not exceed $15'\sqrt{N}$ mins, where N is the number of sides of the traverse.

$$\text{Relative closing error} = \frac{\text{amount of closing error}}{\text{perimeter of traverse}}$$

The value should not exceed 1/600.

3.18 SOURCES OF ERROR IN COMPASS

The following are the kinds of error which may occur while taking readings with a compass:

1. Instrumental errors

(a) The needle may not be perfectly straight and might not be balanced properly.
(b) The pivot point may be eccentric.
(c) The graduations of the ring may not be uniform.
(d) The ring may not rotate freely on account of the pivot point being blunt. This may occur due to the head of the pivot being broken because of careless handling.
(e) The sight vane may not be vertical.
(f) The horse hair may not be straight and vertical.

2. Personal errors

(a) The centring may not be done perfectly over the station.
(b) The graduated ring may not be levelled.
(c) The object might not be bisected properly.
(d) The readings may be taken or entered carelessly.
(e) The observer may be carrying magnetic substances.

3. Other sources of error

(a) There may be local attraction due to the presence of magnetic substances near the station.

(b) The magnetic field could vary on account of some natural causes.
(c) The magnetic declination might vary.

3.19 PRECAUTIONS TO BE TAKEN IN COMPASS SURVEYING

The following precautions should be taken while conducting a compass traverse:

1. The centring should be done perfectly.
2. To stop the rotation of the graduated ring, the brake pin should be pressed very gently and not suddenly.
3. Readings should be taken along the line of sight and not from any side.
4. When the compass has to be shifted from one station to other, the sight vane should be folded over the glass cover. This is done to lift the ring out of the pivot to avoid unnecessary wear of the pivot head.
5. The compass box should be tapped gently before taking the reading. This is done to find out whether the needle rotates freely.
6. The stations should not be selected near magnetic substances.
7. The observer should not carry magnetic substances.
8. The glass cover should not be dusted with a handkerchief, because the glass may be charged with electricity and the needle may be deflected from its true direction. The glass cover should be cleaned with a moist finger.

SHORT QUESTIONS WITH ANSWERS FOR VIVA

Q. 1 What is the principle of compass surveying?
Ans. The principle of compass surveying is traversing, which means that the area is enclosed by series of connected lines. The magnetic bearings of these lines are taken with the compass and the distances of sides are measured by chain.

Q. 2 What is the difference between triangulation and traversing?
Ans. Triangulation involves dividing an area into a number of well-conditioned triangles. But traversing involves the consideration of a series of connected lines.

Q. 3 What does the term 'chain angle' mean?
Ans. When the angle between any two adjacent sides is fixed by chain and tape only by taking tie line. The angle is said to be the chain angle.

Q. 4 What is a 12 cm compass?
Ans. The size of a compass is designated by its diameter. Therefore, a 12 cm compass is a compass of diameter 12 cm.

Q. 5 What is the fundamental difference between the prismatic compass and the surveyor's compass?
Ans. The prismatic compass shows the whole circle bearing of a line, whereas the surveyor's compass shows the quadrantal bearing of a line.

Q. 6 How would you detect the presence of local attraction in an area.
Ans. When the FB and BB of a line differ by exactly 180°, then the line is free from local attraction. The presence of local attraction is established when the FB and BB do not differ by 180°.

Q. 7 The FB of a line is 96°30′ and BB is 276°0′. How will you adjust the bearings?
Ans. Here, FB of line is 96°30′.

So, BB of this line = 96°30′ + 180°0′ = 276°30′

But the observed is 276°0′.

So Adjusted BB = 1/2 (276°30′ + 276°0′) = 276°15′

and Adjusted FB = 276°15′ − 180°0′ = 96°15′

Q. 8 What is local attraction?

Ans. The disturbing influence of magnetic substances on a magnetic needle is known as local attraction.

Q. 9 What is declination?

Ans. The horizontal angle between the true meridian and magnetic meridian is known as declination.

Q. 10 What are isogonic and agonic lines?

Ans. The line passing through points of equal declination is known as the isogonic line, and the one passing through points of zero declination is called the agonic line.

Q. 11 What do you mean by azimuth?

Ans. The true bearing of a line is also known as its azimuth.

Q. 12 The FB of a line is 145°30′. What is its BB?

Ans. BB of the line = 145°30′ + 180°0′ = 325°30′

Q.13 The FB of a line is S 45°30′ W? What is its B.B.?

Ans. BB of the line = N 45°30′ E

Q. 14 What are the precautions to be taken while shifting a prismatic compass from one station to another?

Ans. The sight vane must be folded.

Q. 15 A compass was properly balanced at the equator. What will be the effect on the needle if it is taken to the northern hemisphere?

Ans. The north end of the needle will be inclined towards the North Pole.

Q. 16 What is the angular check of a closed traverse?

Ans. The sum of the interior angles should be equal to $(2N - 4) \times 90°$, where N is the number of sides of traverse.

Q. 17 How would you check the accuracy of open traverse?

Ans. The accuracy of open traverse is checked by taking cut-off lines or an auxiliary point.

EXERCISES

1. What does traverse surveying mean?
   Distinguish between closed and open traverse.
2. What is local attraction?
   How it is detected and adjusted?
3. Describe the field procedure of prismatic compass survey.
4. Describe the plotting of a compass survey map.
5. What is 'closing error' in a traverse?
   Describe, with a sketch, how such an error is adjusted.
6. What could be the sources of error in a compass traverse?
7. What are the precautions to be taken in a compass survey?
8. Explain clearly the difference between a prismatic compass and a surveyor's compass.
9. How can the accuracy of a closed traverse and open traverse be checked?
10. Define the following:
    (a) Whole circle bearing and reduced bearing,
    (b) Fore bearing and back bearing,
    (c) True meridian and magnetic meridian,

    (d) Magnetic declination,
    (e) Dip of the magnetic needle, and
    (f) Local attraction.
11. A line was shown to a magnetic bearing of 35°15′ in an old map, when the declination was 13°45′ E. To what bearing should it be set now if the present magnetic declination is 4°15′ W. (WBSC, 1989)
    (Ans. 53.15′)
12. The following bearings were observed in a closed traverse. Find the included angles and correct the bearings of the lines. Draw the sketch of the plot, if AB = 90 m, BC = 90 m and CD = 60 m and show in it all included angles. (WBSC, 1987)

Line	FB	BB
AB	140°45′	318°15′
BC	216°30′	38°00′
CD	209°15′	30°15′
DE	319°45′	139°45′
EA	60°15′	240°15′

13. The following fore and back bearings were observed while traversing an area with a compass:

Line	FB	BB
PQ	S 37°30′ E	N 37°30′ W
QR	S 43°15′ W	N 44°15′ E
RS	N 73°00′ W	S 72°15′ E
ST	N 12°45′ E	S 13°15′ W
TP	N 60°00′ E	S 59°15′ W

Find the corrected bearing of the line.
(AMIE, Winter, 1988)

14. The following bearings were observed in a compass traverse:

Line	FB	BB
AB	305°00′	125°30′
BC	75°30′	254°30′
CD	115°30′	297°00′
DE	165°30′	345°30′
EA	225°00′	44°00′

At which of these stations would local attraction be suspected? Find the corrected bearings of the lines. Find also the true bearings of the lines, if the magnetic declination is 4°30′ W. (AMIE Summer, 1986)

15. Choose the correct alternative in questions (i) to (xv).
    (i) In a prismatic compass, the zero is marked on the
    (a) North end (b) South end (c) West end
    (ii) In a surveyor's compass, the ring is graduated
    (a) From 0° to 360° (b) In quadrants—0° to 90° (c) in any way
    (iii) The compass box is made of
    (a) Iron (b) Aluminium (c) Brass
    (iv) Open traverse is suitable in the survey of
    (a) Ponds (b) Rivers (c) Estates
    (v) The sum of interior angles of a closed traverse is
    (a) $(2n - 4) \times 90°$ (b) $(2n + 4) \times 90°$ (c) $(n - 4) \times 90°$

(vi) At the equator the dip of the needle is
(a) 180° (b) 0° (c) 90°
(vii) At the magnetic pole, the dip is
(a) 0° (b) 45° (c) 90°
(viii) The true meridian passes through
(a) Geographical poles (b) Magnetic poles (c) Arbitrary poles
(ix) The line passing through 'zero' declination is known as the
(a) Isogonic line (b) Agonic line (c) Contour line
(x) In the WCB system, a line is said to be free from local attraction if the difference between the FB and BB is
(a) 0° (b) 90° (c) 180°
(xi) In the QB system, a line is said to be free from local attraction, if the FB and BB are
(a) numerically equal (b) numerically equal, with opposite quadrants
(c) Anything
(xii) The accuracy of open traverse is checked by the
(a) Cut-off line (b) Auxiliary line (c) Random line
(xiii) The angular error of closure should not exceed
(a) $15'\sqrt{N}$ min (b) $30'\sqrt{N}$ min (c) $\sqrt{N}$ min
(xiv) The closing error in a closed traverse is adjusted by
(a) Lehmann's rule (b) Bowditch's rule (c) Slide rule
(xv) The relative closing error should not exceed
(a) $\frac{1}{600}$ (b) $\frac{1}{400}$ (c) $\frac{1}{1,000}$

ANSWERS

15.	(i) b	(ii) b	(iii) c	(iv) b	(v) a
	(vi) b	(vii) c	(viii) a	(ix) b	(x) c
	(xi) b	(xii) a	(xiii) a	(xiv) b	(xv) a

4

Plane Table Surveying

4.1 PRINCIPLE

The principle of plane tabling is parallelism, meaning that the rays drawn from stations to objects on the paper are parallel to the lines from the stations to the objects on the ground. The relative positions of the objects on the ground are represented by their plotted positions on the paper and lie on the respective rays. The table is always placed at each of the successive stations parallel to the position it occupied at the starting station. Plane tabling is a graphical method of surveying. Here, the field work and plotting are done simultaneously and such survey does not involve the use of a field book.

Plane table survey is mainly suitable for filling interior details when traversing is done by theodolite. Sometimes traversing by plane table may also be done. But this survey is recommended for the work where great accuracy is not required. As the fitting and fixing arrangement of this instrument is not perfect, most accurate work cannot be expected.

4.2 ACCESSORIES OF PLANE TABLE

1. The Plane Table The plane table is a drawing board of size 750 mm × 600 mm made of well-seasoned wood like teak, pine, etc. The top surface of the table is well levelled. The bottom surface consists of a threaded circular plate for fixing the table on the tripod stand by a wing nut.

The plane table is meant for fixing a drawing sheet over it. The positions of the objects are located on this sheet by drawing rays and plotting to any suitable scale (Fig. 4.1).

2. The alidade There are two types of alidade—plain and telescopic.

(a) Plain Alidade The plain alidade consists of a metal or wooden ruler of length about 50 cm. One of its edges is bevelled, and is known as the fiducial edge. It consists of two vanes at both ends which are hinged with the ruler. One is known as the 'object vane' and carries a horse hair; the other is called the 'sight vane' and is provided with a narrow slit (Fig. 4.2).

(b) Telescopic Alidade The telescopic alidade consists of a telescope meant for inclined sight or sighting distant objects clearly. This alidade has no vanes at the ends, but is provided with fiducial edge.

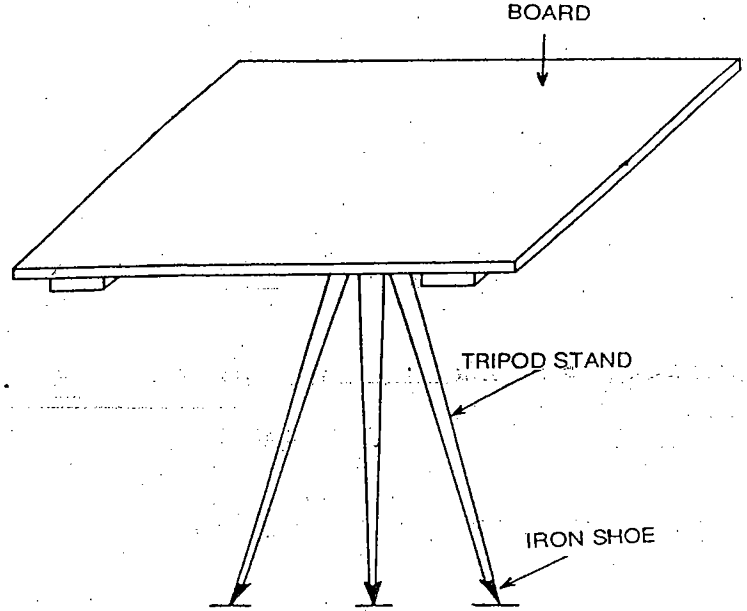


Fig. 4.1

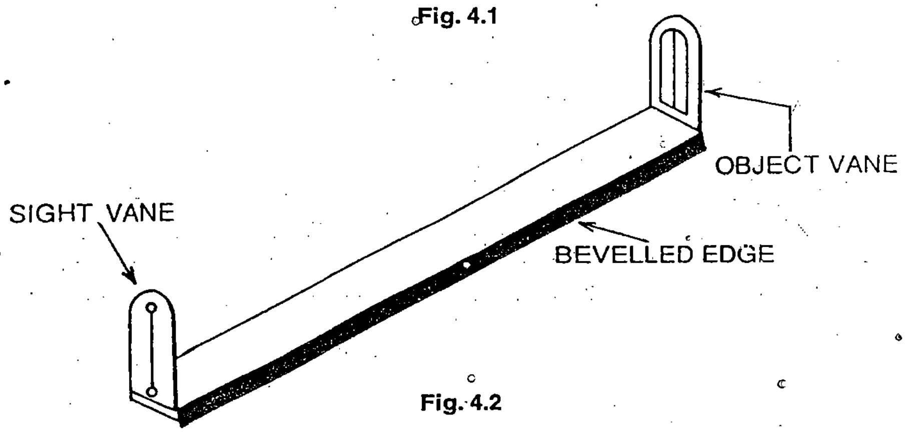


Fig. 4.2

The function of the alidade is to sight objects. The rays should be drawn along the fiducial edge (Fig. 4.3).

3. The spirit Level The spirit level is a small metal tube containing a small bubble of spirit. The bubble is visible on the top along a graduated glass tube.

The spirit level is meant for levelling the plane table (Fig. 4.4).

4. The compass There are two kinds of compass—(a) the trough compass, and (b) the circular box compass.

(a) The Trough Compass The trough compass is a rectangular box made of non-magnetic metal containing a magnetic needle pivoted at the centre. This compass consists of a '0' mark at both ends to locate the N-S direction (Fig. 4.5).

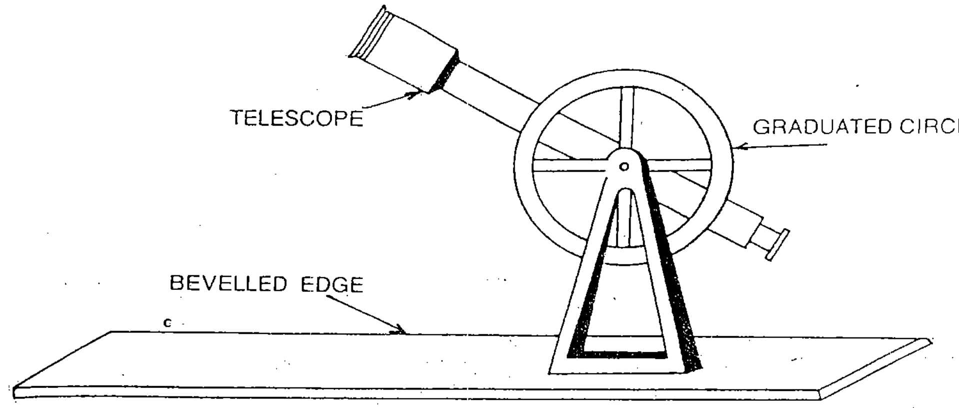


Fig. 4.3

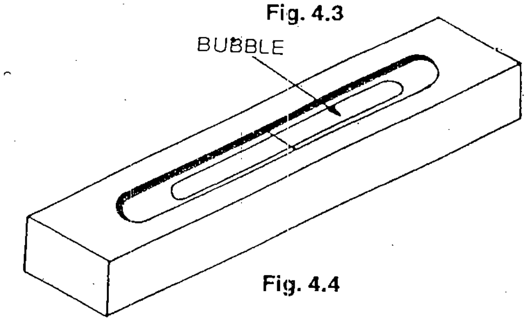


Fig. 4.4

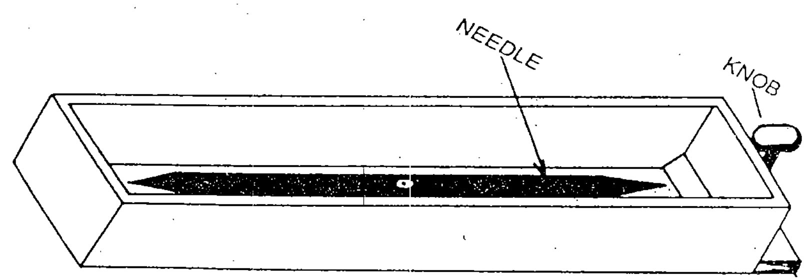


Fig. 4.5

(b) The Circular Box Compass It carries a pivoted magnetic needle at the centre. The circular box is fitted on a square base plate (Fig. 4.6).

Sometimes two bubble tubes are fixed at right angles to each other on the base plate. The compass is meant for marking the north direction of the map.

5. U-fork or plumbing fork with plumb bob The U-fork is a metal strip bent in the shape of a 'U' (hair pin) having equal arm lengths. The top arm is pointed and the bottom arm carries a hook for suspending a plumb bob (Fig. 4.7).

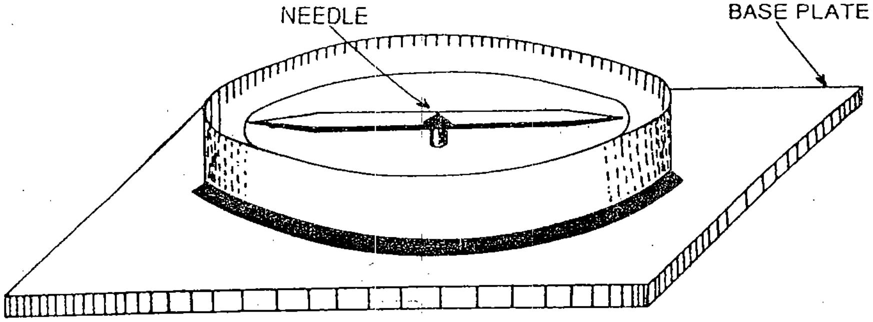


Fig. 4.6

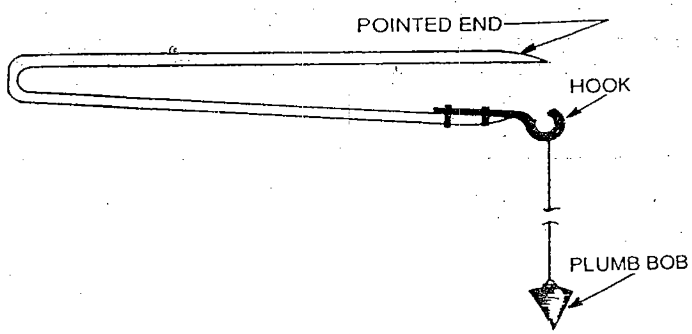


Fig. 4.7

This is meant for centering the table over a station.

4.3 ORIENTATION

The method of setting up the plane table at each of the successive stations parallel to the position it occupied at the starting station is known as orientation.

Orientation must be done when the plane table is set up at more than one station. As already stated, plane tabling is based on the principle of parallelism. So, the relative positions of the objects on the map will be accurate only if the orientation is proper. But if orientation is not done, then the map will not represent the actual positions of the objects.

Orientation may be done by magnetic needle and backsighting.

Orientation by magnetic needle This method is suitable when local attraction is not suspected in the area.

Procedure (a) Suppose A and B are two stations. The plane table is set up at station A, and levelled by spirit level. The centring is done by U-fork and plumb bob so that point *a* is just over station A. Then the trough compass or circular box compass is placed on the right-hand top corner of the sheet in such a way that the needle coincides with '0–0' mark. After this, a line representing the north line is drawn through the edge of the compass box. The table is then clamped.

(b) With the alidade touching the point *a*, the ranging rod at B is bisected and a ray is drawn. The distance AB is measured and plotted to any suitable scale.

(c) The table is shifted and centred over B, so that point *b* is just over B. The table is levelled. Now the trough compass is placed exactly along the north line drawn previously. The table is then turned clockwise or anticlockwise until the needle coincides exactly with the 0–0 mark of the compass.

While turning the table, care should be taken not to disturb the centering. In case it is, it should be adjusted immediately.

(d) When the centring and levelling are perfect and the needle is exactly at 0–0, the orientation is said to be perfect (Fig. 4.8).

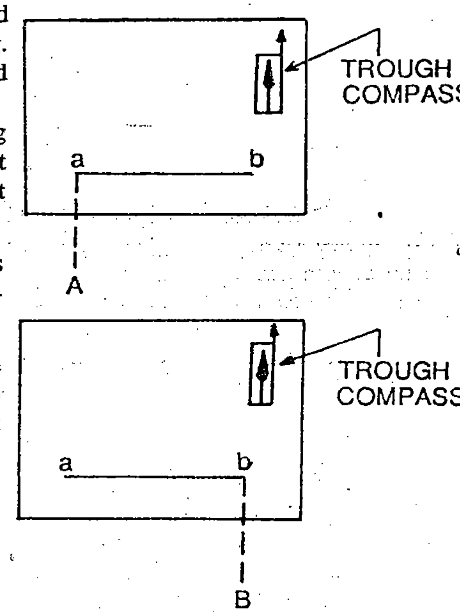


Fig. 4.8

2. Orientation by backsighting This method is accurate and is always preferred.

Procedure (a) Suppose A and B are two stations. The plane table is set up over A. The table is levelled by spirit level and centered by U-fork so that point *a* is just over station A. The north line is marked on the right-hand top corner of the sheet by trough compass.

(b) With the alidade touching *a*, the ranging rod at B is bisected and a ray is drawn. The distance AB is measured and plotted to any suitable scale. So, the point *b* represents station B.

(c) The table is shifted and set up over B. It is levelled and centered so that *b* is just over B. Now the alidade is placed along the line *ba*, and the ranging rod at A is bisected by turning the table clockwise or anticlockwise. At this time the centring may be disturbed, and should be adjusted immediately if required. When the centering, levelling and bisection of the ranging rod at A are perfect, then the orientation is said to be perfect (Fig. 4.9).

4.4 PROCEDURE OF SETTING UP PLANE TABLE OVER A STATION

The following steps have to be performed in order to set up a plane table over a station:

1. Fixing the table on the tripod stand The tripod stand is placed over the required station with its legs well apart. Then the table is fixed on it by wing nut at the bottom.

2. Levelling the table The table is levelled by placing the spirit level at different

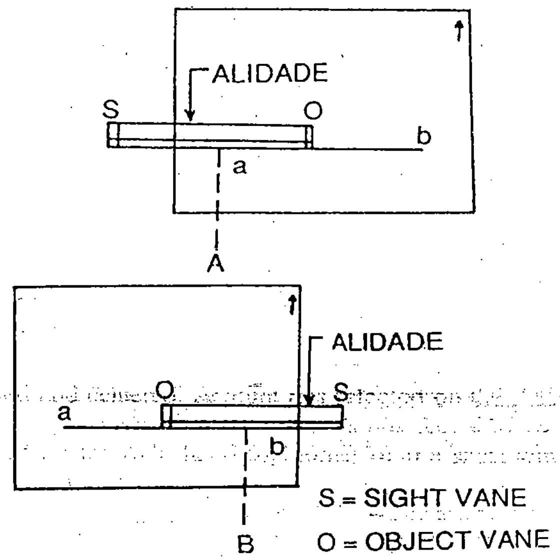


Fig. 4.9

corners and various positions on the table. The bubble is brought to the centre of its run at every positions of the table by adjusting the legs.

3. Centring the table The drawing sheet is fixed on the table. A suitable point P is selected on the sheet to represent the station P on the ground. A pin is then fixed on this selected point. The upper pointed end of the U-fork is made in contact with the station pin and the plumb bob which is suspended from the hook at the lower end is brought just over the station P by turning the table clockwise or anticlockwise or slightly adjusting the legs. This operation is called centring. The table is then clamped. Care should be taken not to disturb the levelling.

4. Marking the north line The trough compass is placed on the right-hand top corner with its north end approximately towards the north. Then the compass is turned clockwise or anticlockwise so that the needle exactly coincides with the 0–0 mark. Now a line representing the north line is drawn through the edge of the compass. It should be ensured that the table is not turned.

5. Orientation When plane table survey is to be conducted by connecting several stations, the orientation must be performed at every successive station. It may be done by magnetic needle or by the backsighting method. The backsighting process is always preferred, because it is reliable. During orientation, it should always be remembered that the requirements of centring, levelling, and orientation must be satisfied simultaneously.

4.5 METHODS OF PLANE TABLING

The following are the four methods of plane tabling:

1. Radiation
2. Intersection
3. Traversing and
4. Resection

1. Radiation This method is suitable for locating the objects from a single station. In this method, rays are drawn from the station to the objects, and the distances from the station to the objects are measured and plotted to any suitable scale along the respective rays.

Procedure (a) Suppose P is a station on the ground from where the objects A, B, C and D are visible.

(b) The plane table is set up over the station P. A drawing sheet is fixed on the table, which is then levelled and centered. A point *p* is selected on the sheet to represent the station P.

(c) The north line is marked on the right-hand top corner of the sheet with trough compass or circular box compass.

(d) With the alidade touching *p*, the ranging rods at A, B, C and D are bisected and the rays drawn.

(e) The distances PA, PB, PC, and PD are measured and plotted to any suitable scale to obtain the points a, b, c, and d, representing the objects A, B, C and D (Fig. 4.10), on paper.

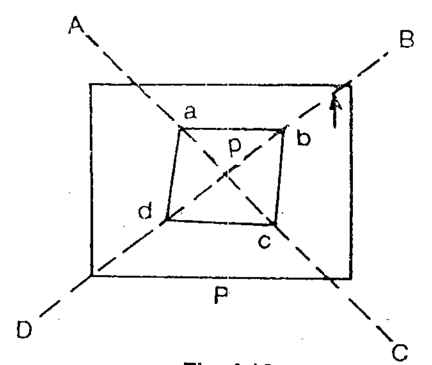


Fig. 4.10

2. The Intersection method This method is suitable for locating inaccessible points by the intersection of the rays drawn from two instrument stations.

Procedure (a) Suppose A and B are two stations and P is an object on the far bank of a river. Now it is required to fix the position of P on the sheet by the intersection of rays, drawn from A and B.

(b) The table is set up at A. It is levelled and centred so that a point *a* on the sheet is just over the station A. The north line is marked on the right-hand top corner. The table is then clamped.

(c) With the alidade touching *a*, the object P and the ranging rod at B are bisected, and rays are drawn through the fiducial edge of the alidade.

(d) The distance AB is measured and plotted to any suitable scale to obtain the point *b*.

(e) The table is shifted and centred over B and levelled properly. Now the alidade is placed along the line ba and orientation is done by backsighting. At this time it should be remembered that the centring, levelling and orientation must be perfect simultaneously.

(f) With the alidade touching *b*, the object P is bisected and a ray is drawn. Suppose this ray intersects the previous ray at a point *p*. This point *p* is the required plotted position of P (Fig. 4.11).

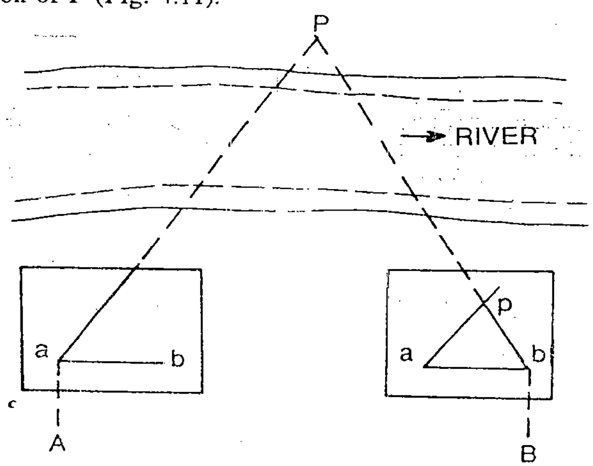


Fig. 4.11

3. The traversing method This method is suitable for connecting the traverse stations. This is similar to compass traversing or theodolite traversing. But here fielding and plotting are done simultaneously with the help of the radiation and intersection methods.

Procedure (a) Suppose A, B, C and D are the traverse stations.

(b) The table is set up at the station A. A suitable point *a* is selected on the sheet in such a way that the whole area may be plotted in the sheet. The table is centred, levelled and clamped. The north line is marked on the right-hand top corner of the sheet.

(c) With the alidade touching point *a* the ranging rod at B is bisected and a ray is drawn. The distance AB is measured and plotted to any suitable scale.

(d) The table is shifted and centred over B. It is then levelled, oriented by back-sighting and clamped.

(e) With the alidade touching point *b*, the ranging rod at C is bisected and a ray is drawn. The distance BC is measured and plotted to the same scale.

(f) The table is shifted and set up at C and the same procedure is repeated.

(g) In this manner, all stations of the traverse are connected.

(h) At the end, the finishing point may not coincide with the starting point and there may be some closing error. This error is adjusted graphically by Bowditch's rule (given in Sec. 3.13).

(i) After making the corrections for closing error, the table is again set up at A. After centering, levelling and orientation, the surrounding details are located by radiation.

(j) The table is then shifted and set up at all the stations of the traverse and after proper adjustments the details are located by the radiation and intersection methods (Fig. 4.12).

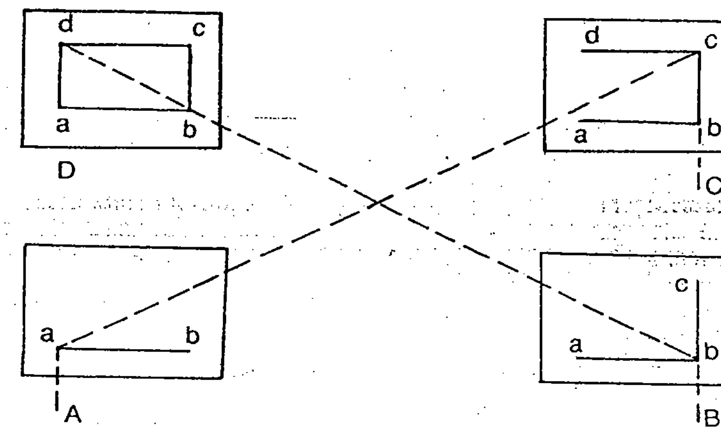


Fig. 4.12

4. The resection method This method is suitable for establishing new stations at a place in order to locate missing details.

Procedure (a) Suppose it is required to establish a station at position P. Let us select two points A and B on the ground. The distance AB is measured and plotted to any suitable scale. This line AB is known as the "base line".

(b) The table is set up at A. It is levelled, centred and oriented by bisecting the ranging rod at B. The table is then clamped.

(c) With the alidade touching point *a*, the ranging rod at P is bisected and a ray is drawn. Then a point P_1 is marked on this ray by estimating with the eye.

(d) The table is shifted and centred in such a way that P_1 is just over P. It is then oriented by backsighting the ranging rod at A.

(e) With the alidade touching point *b*, the ranging rod at B is bisected and a ray is drawn. Suppose this ray intersects the previous ray at a point P. This point represents the position of the station P on the sheet. Then the actual position of the station P is marked on the ground by U-fork and plumb bob (Fig. 4.13).

4.6 SPECIAL METHODS OF RESECTION

Sometimes, after the completion of plane table traversing, it may be noticed that an important object has not been located due to oversight. If no station pegs are found on the field, some special methods of resection are applied in order to establish a new station for plotting the missing object. The methods are based on: (1) the two-point problem, and (2) the three-point problem.

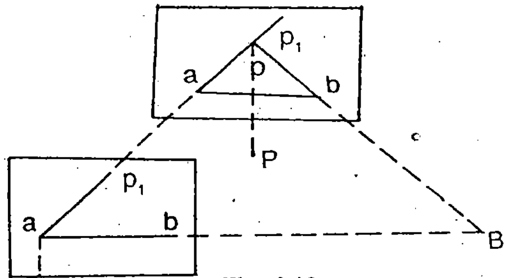


Fig. 4.13

1. The two-point problem In this problem, two well-defined points whose positions have already been plotted on the plan are selected. Then, by perfectly bisecting these points, a new station is established at the required position.

Procedure (a) Suppose P and Q are two well-defined points whose positions are plotted on map as p and q. It is required to locate a new station at A by perfectly bisecting P and Q.

(b) An auxiliary station B is selected at a suitable position. The table is set up at B, and levelled and oriented by eye estimation. It is then clamped.

(c) With the alidade touching p and q, the points P and Q are bisected and rays are drawn. Suppose these rays intersect at b.

(d) With the alidade centred on b, the ranging rod at A is bisected and a ray is drawn. Then, by eye estimation, a point a_1 is marked on this ray.

(e) The table is shifted and centred on A, with a_1 just over A. It is levelled and oriented by backsighting. With the alidade touching p, the point P is bisected and a ray is drawn. Suppose this ray intersects the line ba_1 at point a_1, as was assumed previously.

(f) With the alidade centered on a_1 the point Q is bisected and a ray is drawn. Suppose this ray intersects the ray bq at a point q_1. The triangle pqq_1 is known as the triangle of error, and is to be eliminated.

(g) The alidade is placed along the line pq_1 and a ranging rod R is fixed at some distance from the table. Then, the alidade is placed along the line pq and the table is turned to bisect R. At this position the table is said to be perfectly oriented.

(h) Finally, with the alidade centred on p and q, the points P and Q are bisected and rays are drawn. Suppose these rays intersect at a point a. This would represent the exact position of the required station A (Fig. 4.14). Then the station A is marked on the ground.

2. The Three-point problem In this problem, three well-defined points are selected whose positions have already been plotted on the map. Then, by perfectly bisecting these three well-defined points, a new station is established at the required position.

No auxiliary station is required in order to solve this problem. The table is directly placed at the required position. The problem may be solved by three methods: (a) the graphical or Bessel's method, (b) the mechanical method, and (c) the trial-and-error method.

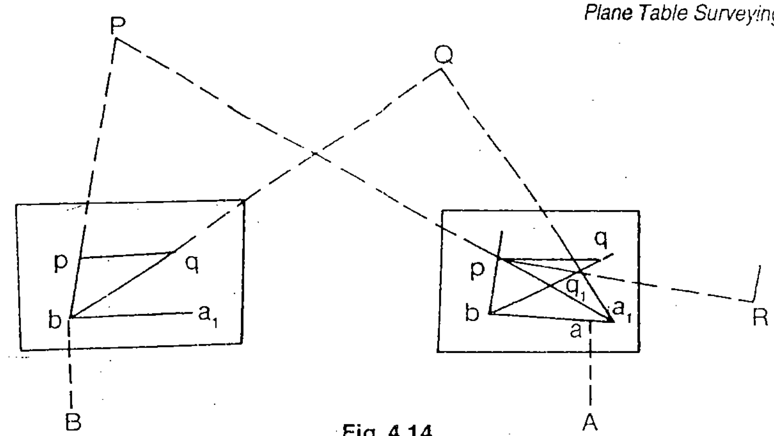


Fig. 4.14

(a) The Graphical Method (i) Suppose A, B and C are three well-defined points which have been plotted as a, b, and c. Now it is required to locate a station at P.

(ii) The table is placed at the required station P and levelled. The alidade is placed along the line ca and the point A is bisected. The table is clamped. With the alidade centred on C, the point B is bisected and ray is drawn (Fig. 4.15(a)).

(iii) Again the alidade is placed along the line ac and the point C is bisected and the table is clamped. With the alidade touching a, the point B is bisected and a ray is drawn. Suppose this ray intersects the previous ray at a point d (Fig. 4.15(b)).

(iv) The alidade is placed along db and the point B is bisected. At this position the table is said to be perfectly oriented. Now the rays Aa, Bb and Cc are drawn. These three rays must meet at a point p which is the required point on the map. This point is transferred to the ground by U-fork and plumb-bob (Fig. 4.15(c)).

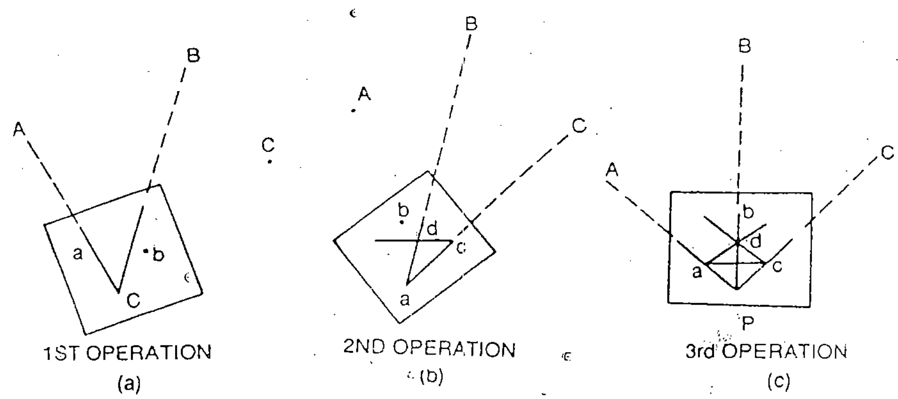


Fig. 4.15

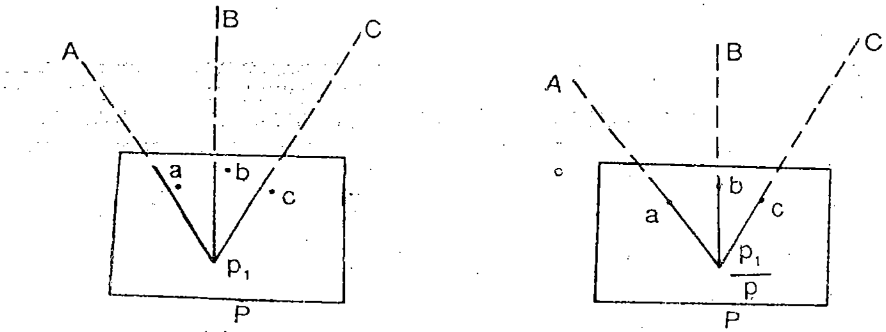
(b) The Mechanical Method (i) Suppose A, B and C are three well-defined points which have been plotted on the map as a, b, and c. It is required to locate a station at P.

(ii) The table is placed at P and levelled. A tracing paper is fixed on the map and a point *p* is marked on it.

(iii) With the alidade centred on P the points A, B and C are bisected and rays are drawn. These rays may not pass through the points a, b and c as the orientation is done approximately (Fig. 4.16(a)).

(iv) Now the tracing paper is unfastened and moved over the map in such a way that the three rays simultaneously pass through the plotted positions a, b, and c. Then the point *p* is pricked with a pin to give an impression *p* on the map. *p* is the required point on the map. The tracing paper is then removed (Fig. 4.16(b)).

(v) Then the alidade is centred on *p* and the rays are drawn towards A, B and C. These rays must pass through the points *a*, *b* and *c*.



(a) Fig. 4.16 (b)

(c) The Method of Trial and Error

Procedure (a) Suppose A, B and C are three well-defined points which have been plotted as a, b, and c on the map. Now it is required to establish a station at P.

(b) The table is set up at P and levelled. Orientation is done by eye estimation.

(c) With the alidade, rays Aa, Bb and Cc are drawn. As the orientation is approximate, the rays may not intersect at a point, but may form a small triangle—the triangle of error.

(d) To get the actual point, this triangle of error is to be eliminated. By repeatedly turning the table clockwise or anticlockwise, the triangle is eliminated in such a way that the rays Aa, Bb, and Cc finally meet at a point *p*. This is the required point on the map. This point is transferred to the ground by U-fork and plumb bob (Fig. 4.17).

4.7 ERRORS AND PRECAUTIONS

The following are the common errors in plane tabling:

A. Instrumental Errors

1. The surface of the table may not be perfectly level.
2. The fiducial edge of the alidade might not be straight.

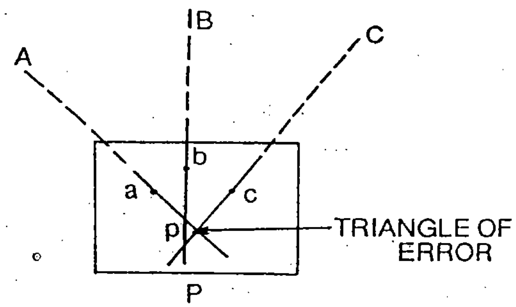


Fig. 4.17

3. The vanes may not be vertical.
4. The horsehair may be loose and inclined.
5. The table may be loosely joined with the tripod stand.
6. The needle of the trough compass may not be perfectly balanced. Also it may not be able to move freely due to sluggishness of the pivot point.

The above errors are adjustable or avoidable.

B. Personal Errors

1. The levelling of the table may not be perfect.
2. The table may not be centred properly.
3. The orientation of the table may not be proper.
4. The table might not be perfectly clamped.
5. The objects may not be bisected perfectly.
6. The alidade may not be correctly centred on the station point.
7. The rays might not be drawn accurately.
8. The alidade may not be centred on the same side of the station point throughout the work.

C. Plotting Error

1. A good quality pencil with a very fine pointed end may not have been used.
2. An incorrect scale may be used by mistake.
3. Errors may result from failure to observe the correct measurement from the scale.
4. Unnecessary hurry at the time of plotting may lead to plotting errors.

The following precautions should be taken while using the plane table:

1. Before starting the work the equipments for survey work should be verified. Defective accessories should be replaced by perfect equipment.
2. The centring should be perfect.
3. The levelling should be proper.
4. The orientation should be accurate.

5. The alidade should be centred on the same side of the station-pin until the work is completed.
   (*Note*: If the work was started with the alidade on the left of the station-pin, then the work should be completed by maintaining this position. This is the ideal position of the alidade and should always be preferred.)
6. While shifting the plane table from one station to another, the tripod stand should be kept vertical to avoid damage to the fixing arrangement.
7. Several accessories have to be carried. So, care should be taken to ensure that nothing is missing.
8. The pencil should have a sharp point.
9. The distances of the objects or lines should be written temporarily along the respective rays until the plotting is completed.
10. Only the selected scale should be on the table.
11. Measurements should be taken carefully from the scale while plotting.
12. The stations on the ground are marked A, B, C, D, etc. while the station points on the map are marked a, b, c, d etc.

4.8 PROCEDURE OF PLANE TABLE TRAVERSING

A. Equipment Required

1.	Plane table with tripod stand	—	1 no.
2.	Alidade (plain or telescopic)	—	1 no.
3.	Trough compass or circular box compass	—	1 no.
4.	Spirit level	—	1 no.
5.	U-fork with plumb bob	—	1 set
6.	Metric chain (20 m)	—	1 no.
7.	Metallic tape (15 m)	—	1 no.
8.	Arrows	—	10 nos
9.	Ranging rods	—	3 nos
10.	Wooden pegs	—	10 nos.
11.	Mallet	—	1 no.
12.	Drawing sheet (good quality)	—	1 no.
13.	Board pins or clips	—	4 nos
14.	Cardboard scale (set of 8 scales)	—	1 set
15.	Good pencil	—	1 no.
16.	Eraser	—	1 no.
17.	Knife	—	1 no.
18.	Pins	—	5 nos
19.	Set-square (45°, 60°)	—	2 nos
20.	Duster	—	1 no.

B. Procedure of Field Work

1. Reconnaissance The area to be surveyed is thoroughly examined to find the best possible way for traversing. The traverse stations should cover the whole area and should be intervisible. The provisions for check lines should be kept in mind.

2. Marking the stations The selected stations are marked on the ground by wooden pegs. Reference sketches should be prepared for the stations so that they can be readily located in case the station pegs are removed.

3. Connecting the traverse legs and marking details A detailed description is given in Sec. 4.5. Simply follow the procedure outlined there.

4.9 ADVANTAGES AND DISADVANTAGES OF PLANE TABLING

A. Advantages

1. It is the most rapid method of surveying.
2. There is no need for a field book as plotting is done along with the field work. So, the problem of mistakes in booking field notes does not arise.
3. Plotted work can be compared with actual object regardless of whether or not they are properly represented.
4. There is no possibility of overlooking any important object.
5. There is no possibility of overlooking any measurement as plotting is done in the field.
6. Irregular objects may be represented accurately.
7. It is suitable in magnetic areas.
8. The map can be prepared easily, and does not require any great skill.
9. Errors in measurement and plotting can be detected by check lines.
10. Inaccessible points can be easily located by intersection.

B. Disadvantages

1. The plane table is not suitable for accurate work as the fitting arrangement is not perfect.
2. Plane table surveying is not suitable in wet climate, in the rainy season, on foggy mornings and in windy weather.
3. The number of accessories required in such survey is large, and they are likely to be lost.
4. The instrument is very heavy and difficult to carry.
5. The map cannot be replotted to a different scale as there is no field book.

SHORT QUESTIONS WITH ANSWERS FOR VIVA

Q. 1 What is the principle of plane tabling?

Ans. The principle of plane tabling is parallelism, meaning that the plane table is always placed in every station parallel to the position it occupied at the first station.

Q. 2 What is orientation? Why is it done?

Ans. The method of keeping the table in successive stations parallel to the position it occupied at the starting station is known as orientation. Orientation is done to maintain, perfectly the relative positions of different objects on the map.

Q. 3 How are centring and levelling done in plane tabling?

Ans. The centring is done by U-fork and plumb bob. The upper pointed end of the

U-fork is kept in contact with the station-pin when the plumb bob is just over the station peg.

Levelling is done by spirit level. The spirit level is placed at the different corners and at various positions on the table. By adjusting the legs of the table, the bubble is brought to the centre.

Q. 4 What are the methods of plane tabling?

Ans. The methods of plane tabling are radiation, intersection, traversing and resection.

Q. 5 When would you apply resection?

Ans. To establish a new station with the help of two points or stations.

Q. 6 What is intersection? When it is required?

Ans. The method of locating an object by the intersection of rays drawn from two stations is called the intersection method.

This method is applied for locating inaccessible points, that is, when it is not possible to measure the distance from the station to the object.

Q. 7 When would you apply the two-point and three-point problem?

Ans. If it is found after completion of the plane table survey that an important object has not been plotted, then the two-point or three point problem is applied to locate a new station. These problems can be applied even if all the station pegs have been removed.

Q. 8 What do the terms 'great triangle' and 'great circle' mean?

Ans. In the three-point problem, the triangle formed by joining three well-defined points is known as the great triangle and the circle passing through them is called the great circle.

Q. 9 One month after the completion of a plane table survey, it is detected that one important object was not plotted. How will you plot the object on going to the field?

Ans. A new station has to be established with the help of the two-point or three-point problem. The object can then be located with reference to that new station.

Q. 10 What type of orientation would you prefer and why?

Ans. Orientation by backsighting is always preferred, because it is more reliable than the magnetic needle method. In this method, magnetic substances do not affect the work.

Q. 11. What method would you apply for locating inaccessible points?

Ans. The method of intersection should be applied in order to locate inaccessible points.

Q. 12 What do you mean by the 'fiducial edge' of the alidade?

Ans. The working bevelled edge of the alidade is known as the fiducial edge.

Q. 13 What are the different types of alidade?

Ans. There are two types of alidade—plain and telescopic.

Q. 14 What do you mean by 'strength of fix'?

Ans. In the three-point problem, the relative positions of A, B, C and the required point P should be such that the required point can be located very quickly and accurately. The accuracy with which the point can be fixed is termed the strength of fix.

Q. 15 What are the precautions you have to take while centring the alidade with the station pin?

Ans. The alidade should be centred on the same side of the station pin throughout the traverse. Keeping the alidade on the left of station pin is ideal.

EXERCISES

1. Differentiate between radiation and intersection.

2. What is a two-point problem?
   Explain with a neat sketch the procedure of solving a two-point problem in plane table surveying.
3. Describe the procedure of setting up the plane table over a station.
4. What is orientation? What are the methods of orientation? Describe the methods with a sketch.
5. (a) What is the principle of plane table survey?
   (b) Name the different instruments and accessories used in plane table survey.
   (c) Explain with a sketch the method of radiation.
   (d) What is the basic difference between radiation and intersection.
6. What are the methods of plane tabling? Describe any of them with a sketch.
7. What is a three-point problem? Describe how it is solved by Bessel's method.
8. What are the errors that may occur in plane tabling?
9. What are the precautions to be taken in plane table surveying.
10. What are the advantages and disadvantages of plane tabling?
11. Describe with a sketch how plane table traversing is done.
12. Choose the correct alternative for questions (i) to (ix).
    (i) In plane table survey, the operation which must be carried out is
    (a) Resection (b) Orientation (c) Intersection
    (ii) The working edge of the alidade is known as the
    (a) Fiducial edge (b) Bevelled edge (c) Parallel edge
    (iii) The north line of the map is marked on the
    (a) Right-hand bottom corner (b) Left-hand top corner
    (c) Right-hand top corner
    (iv) The U-fork and plumb bob are required for
    (a) Centring (b) Levelling (c) Orientation
    (v) Inaccessible points may be located by the
    (a) Resection method (b) Intersection method
    (c) Radiation method
    (vi) The accuracy with which the instrument station can be established is known as the
    (a) Strength of fix (b) Strength of accuracy
    (c) Strength of solution
    (vii) The principle of plane table is
    (a) Parallelism (b) Triangulation (c) Traversing
    (viii) The plane table map cannot be plotted to a different scale, as there is no
    (a) Log book (b) Level book (c) Field book
    (ix) The strength of fix is bad, if the plane table station is taken
    (a) Outside the great triangle. (b) On the circumference of the great circle
    (c) Inside the great circle.

ANSWERS

12.	(i) b	(ii) a	(iii) c	(iv) a	(v) b
	(vi) a	(vii) a	(viii) c	(ix) b	

5

Levelling

5.1 OBJECT AND USE OF LEVELLING

Object The aim of levelling is to determine the relative heights of different objects on or below the surface of the earth and to determine the undulation of the ground surface.

Uses Levelling is done for the following purposes:

1. To prepare a contour map for fixing sites for reservoirs, dams, barrages, etc., and to fix the alignment of roads, railways, irrigation canals, and so on.
2. To determine the altitudes of different important points on a hill or to know the reduced levels of different points on or below the surface of the earth.
3. To prepare a longitudinal section and cross-sections of a project (roads, railways, irrigation canals, etc) in order to determine the volume of earth work.
4. To prepare a layout map for water supply, sanitary or drainage schemes.

5.2 DEFINITIONS

1. Levelling The art of determining the relative heights of different points on or below the surface of the earth is known as levelling. Thus, levelling deals with measurements in the vertical plane.

2. Level surface Any surface parallel to the mean spheroidal surface of the earth is said to be a level surface. Such a surface is obviously curved. The water surface of a still lake is also considered to be a level surface.

3. Level line Any line lying on a level surface is called a level line. This line is normal to the plumb line (direction of gravity) at all points (Fig. 5.1).

4. Horizontal plane Any plane tangential to the level surface at any point is known as the horizontal plane. It is perpendicular to the plumb line which indicates the direction of gravity.

5. Horizontal line Any line lying on the horizontal plane is said to be a horizontal line. It is a straight line tangential to the level line (Fig. 5.1).

6. Vertical line The direction indicated by a plumb line (the direction of gravity) is known as the vertical line. This line is perpendicular to the horizontal line (Fig. 5.1).

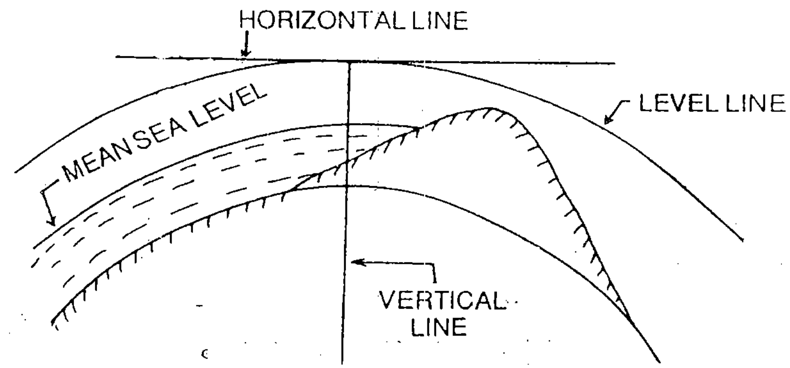


Fig. 5.1 Different Lines

7. Vertical plane Any plane passing through the vertical line is known as the vertical plane.

8. Datum surface or line This is an imaginary level surface or level line from which the vertical distances of different points (above or below this line) are measured. In India the datum adopted for the Great Trigonometrical Survey (GTS) is the mean sea level (MSL) at Karachi.

9. Reduced level (RL) The vertical distance of a point above or below the datum line is known as the reduced level (RL) of that point. The RL of a point may be positive or negative according as the point is above or below the datum.

10. Line of collimation It is an imaginary line passing through the intersection of the cross-hairs at the diaphragm and the optical centre of the object glass and its continuation. It is also known as the line of sight.

11. Axis of the telescope This axis is an imaginary line passing through the optical centre of the object glass and the optical centre of the eye-piece.

12. Axis of bubble tube It is an imaginary line tangential to the longitudinal curve of the bubble tube at its middle point.

13. Bench-marks (BM) These are fixed points or marks of known RL determined with reference to the datum line. These are very important marks. They serve as reference points for finding the RL of new points or for conducting levelling operations in projects involving roads, railways, etc.

Bench-marks may be of four types: (a) GTS, (b) permanent, (c) temporary, and (d) arbitrary.

(a) GTS Bench-marks These bench-marks are established by the Survey of India Department at a large intervals all over the country. The values of reduced levels, the relevant positions and the number of bench-marks are given in a catalogue published by this department (Fig. 5.2).

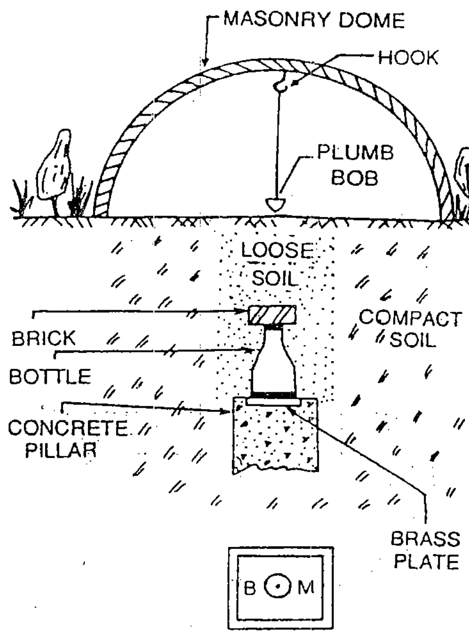


Fig. 5.2

(b) Permanent Bench-marks These are fixed points or marks established by different Government departments like PWD, Railways, Irrigation, etc. The RLs of these points are determined with reference to the GTS bench-mark, and are kept on permanent points like the plinth of a building, parapet of a bridge or culvert, and so on. Sometimes they are kept on underground pillars as shown in Fig. 5.3.

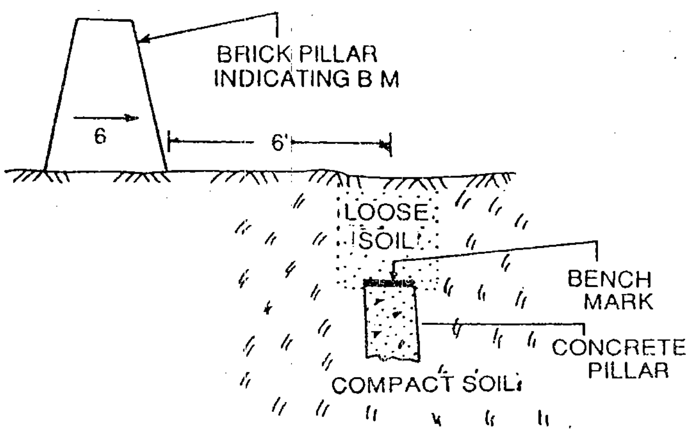


Fig. 5.3

(c) Arbitrary Bench-marks When the RLs of some fixed points are assumed, they are termed arbitrary bench-marks. These are adopted in small survey operations when only the undulation of the ground surface is required to be determined.

(d) Temporary Bench-marks When the bench-marks are established temporarily at the end of a day's work; they are said to be temporary bench-marks. They are generally made on the root of a tree, the parapet of a nearby culvert, a furlong post, or on a similar place.

14. Backsight reading (BS) This is the first staff reading taken in any set up of the instrument after the levelling has been perfectly done. This reading is always taken on a point of known RL, i.e. on a bench-mark or change point (Fig. 5.4).

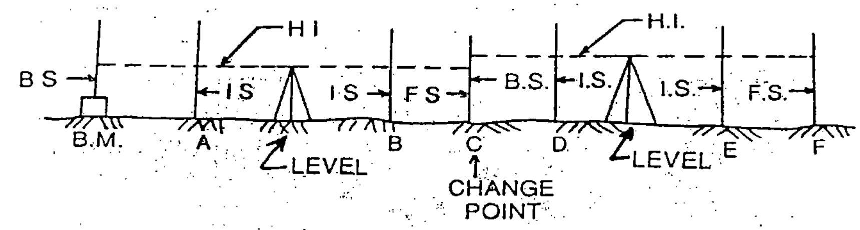


Fig. 5.4

15. Foresight reading (FS) It is the last staff reading in any set up of the instrument, and indicates the shifting of the latter (Fig. 5.4).

16. Intermediate sight reading (IS) It is any other staff reading between the BS and FS in the same set up of the instrument (Fig. 5.4).

17. Change point (CP) This point indicates the shifting of the instrument. At this point, an FS is taken from one setting and a BS from the next setting (Fig. 5.4).

18. Height of instrument (HI) When the levelling instrument is properly levelled, the RL of the line of collimation is known as the height of the instrument. This is obtained by adding the BS reading to the RL of the BM or CP on which the staff reading was taken.

19. Focussing The operation of setting the eye-piece and the object glass a proper distance apart for clear vision of the object is known as focussing. This is done by turning the focussing screw clockwise or anticlockwise.

The function of the object glass is to bring the object into focus on the diaphragm, and that of the eye-piece is to magnify the cross-hairs and object.

Focussing is done in two steps as follows.

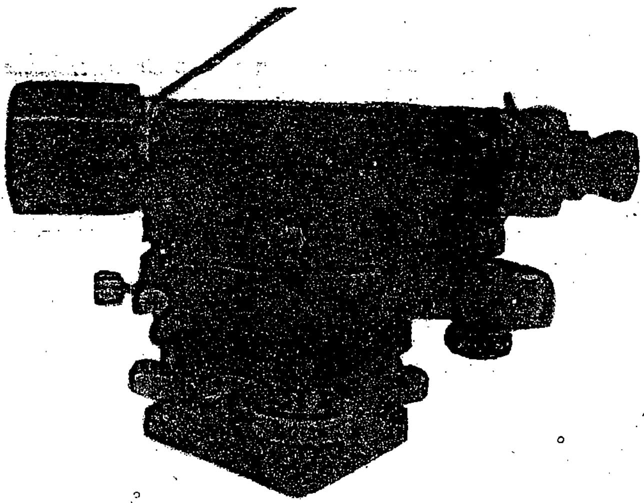
(a) Focussing the eye-piece: A sheet of white paper is held in front of the telescope and the eye-piece is turned clockwise or anticlockwise slowly until the cross-hairs appear distinct and clear.
(b) Focussing the object glass: The telescope is directed to the object and the focussing screw is turned clockwise or anticlockwise until the image is clear and sharp.

20. Parallax The apparent movement of the image relative to the cross-hairs is known as parallax. This occurs due to imperfect focussing, when the image does not fall in the plane of the diaphragm.

The parallax is tested by moving the eye up and down. If the focussing is not correct, the image moves up and down relative to the cross-hairs. If the focussing is perfect, the image appears fixed to the cross-hairs. The parallax may be eliminated by properly focussing the telescope.

5.3 DIFFERENT TYPES OF LEVELS

The following are the different types of level:



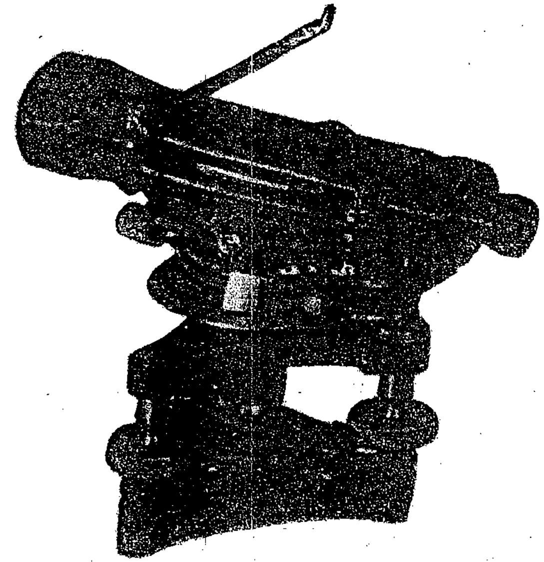
Photograph A Compact Tilting Level

1. The dumpy level The telescope of the dumpy level is rigidly fixed to its supports. It cannot be removed from its supports nor can it be rotated about its longitudinal axis. The instrument is stable and retains its permanent adjustment for a long time. This instrument is commonly used.

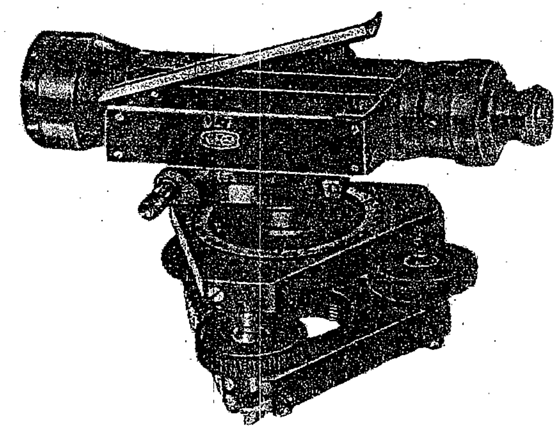
2. The wye level (Y-level) The telescope is held in two 'Y' supports. It can be removed from the supports and reversed from one end of the telescope to the other end. The 'Y' supports consist of two curved clips which may be raised. Thus the telescope can be rotated about its longitudinal axis.

3. Cooke's reversible level This is a combination of the dumpy level and the Y-level. It is supported by two rigid sockets. The telescope can be rotated about its longitudinal axis, withdrawn from the socket and replaced from one end of the telescope to the other end.

4. Cushing's level The telescope cannot be removed from the sockets and rotated about its longitudinal axis. The eye-piece and object glass are removable and can be interchanged from one end of the telescope to the other end.



Photograph Quick-Setting Precision Level



Photograph A Compact Dumpy Level

5. The modern tilting level The telescope can be tilted slightly about its horizontal axis with the help of a tilting screw. In this instrument the line of collimation is made horizontal for each observation by means of the tilting screw.

6. The automatic level This is also known as the self-aligning level. This instrument is levelled automatically within a certain tilt range by means of a compensating device (the tilt compensator).

A. Description of Dumpy Level

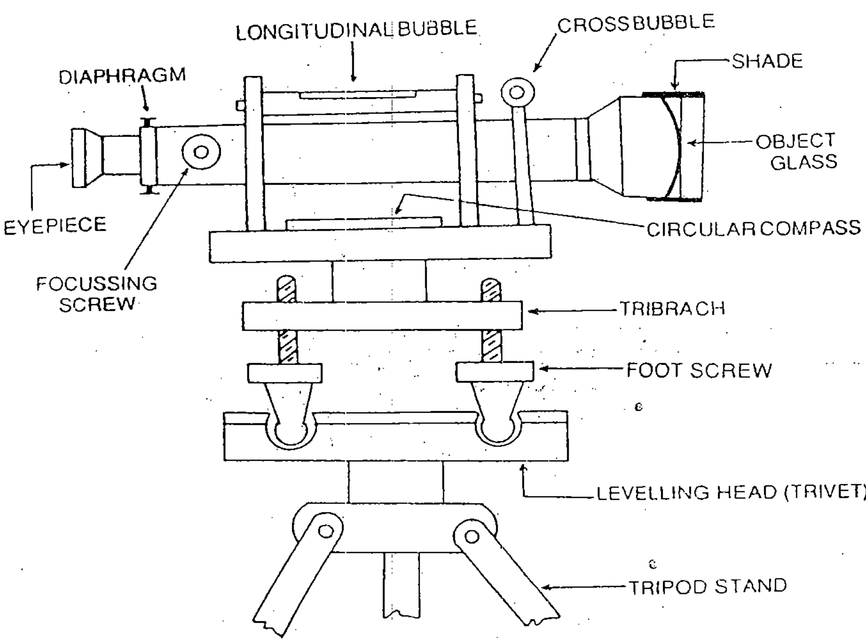


Fig. 5.5

1. Tripod stand The tripod stand consists of three legs which may be solid or framed. The legs are made of light and hard wood. The lower ends of the legs are fitted with steel shoes.

2. Levelling head The levelling head consists of two parallel triangular plates having three grooves to support the foot screws.

3. Foot screws Three foot screws are provided between the trivet and tribrach. By turning the foot screws the tribrach can be raised or lowered to bring the bubble to the centre of its run.

4. Telescope The telescope consists of two metal tubes, one moving within the other. It also consists of an object glass and an eye-piece on opposite ends. A diaphragm is fixed with the telescope just in front of the eye-piece. The diaphragm carries cross-hairs. The telescope is focussed by means of the focussing screw and may have either external focussing, or internal focussing.

In the external focussing telescope, the diaphragm is fixed to the outer tube and the objective to the inner tube. By turning the focussing screw the distance between the objective and diaphragm is altered to form a real image on the plane of cross-hairs.

In the internal focussing telescope, the objective and eye-piece do not move when the focussing screw is turned. Here, a double concave lens is fitted with rack and pinion arrangement between the eye-piece and the objective. This lens moves to and fro when the focussing screw is turned and a real image is formed on the plane of cross-hairs.

5. Bubble tubes Two bubble tubes, one called the longitudinal-bubble tube and other the cross-bubble tube, are placed at right angles to each other. These tubes contain spirit bubble. The bubble is brought to the centre with the help of foot screws. The bubble tubes are fixed on top of the telescope.

6. Compass A compass is provided just below the telescope for taking the magnetic bearing of a line when required.

The compass is graduated in such a way that a 'pointer', which is fixed to the body of compass, indicates a reading of 0° when the telescope is directed along the north line.

In some compasses, the pointer shows a reading of a few degrees when the telescope is directed towards the north. This reading should be taken as the initial reading. The bearing is obtained by deducting the initial reading from the final reading of the compass.

B. Levelling Staff

The levelling staff is a graduated wooden rod used for measuring the vertical distances between the points on the ground and the line of collimation. Levelling staves are classified into two groups: (i) the target staff, and (ii) the self-reading staff.

1. Target staff The target staff consists of a movable target. The target is provided with a vernier which is adjusted by the staffman, according to directions from the levelman, so that the target coincides with the collimation hair. After this, the reading is taken by either the staffman or the levelman. This staff is used for long sightings.

2. Self-reading staff The following are the different types of self-reading staffs:

(a) Sop-with Telescopic Staff Such a staff is arranged in three lengths placed one into the other. It can be extended to its full length by pulling. The top portion is solid and of length 1.25 m, the central box portion is hollow and of length 1.25 m, and the bottom box portion is hollow and 1.5 m long. The total length of the staff is 4 m. The top portions are held in the vertical position by a brass spring catcher.

The staff is graduated in such a way that smallest division is of 5 mm (0.005 m). The values in metres are marked in red on the left and those in decimetres are marked in black on the right (Fig. 5.6).

(b) Folding Metric Staff This staff is made of well-seasoned timber, and is of width 75 mm, thickness 18 mm, and length 4 m. It is divided into two parts of length 2 m having a locking arrangement. It can be folded or detached when required. It is graduated like the telescopic staff (Fig. 5.7).

(c) One-Length Staff The one-length staff, is solid and made of seasoned timber. It is 3 m long and graduated in the same way as the telescopic staff.

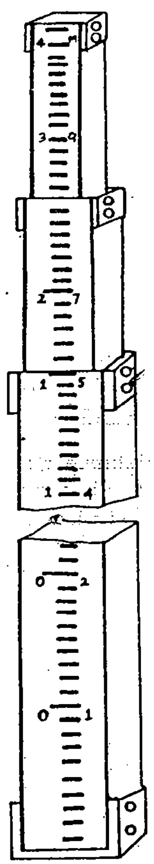


Fig. 5.6 Telescopic Metric Staff

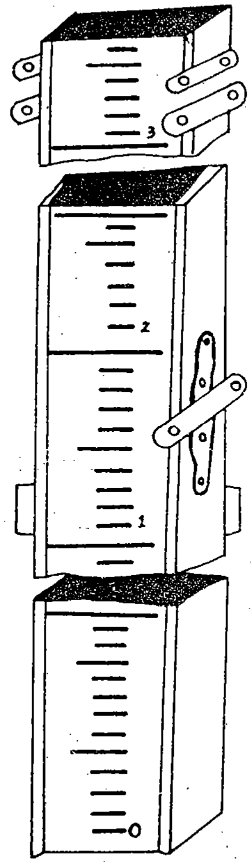


Fig. 5.7 Folding Metric Staff

(d) Invar Staff The invar staff is also 3 m long. An invar band is fitted to a wooden staff. The band is graduated in millimetres. It is used for precise levelling work.

C. Diaphragm

The diaphragm is a brass ring fitted inside the telescope, just in front of the eye-piece. It can be adjusted by four screws. The ring carries the cross-hairs, which get magnified when viewed through the eye-piece. The cross-hairs may be marked in the following ways:

1. With spider webs stretched across the ring,
2. By very fine scratch marks made in a glass fitted with the ring, or
3. By means of platinum wires or silk threads stretched across the ring (Fig. 5.8(a)).

The cross-hair consists of the following lines:

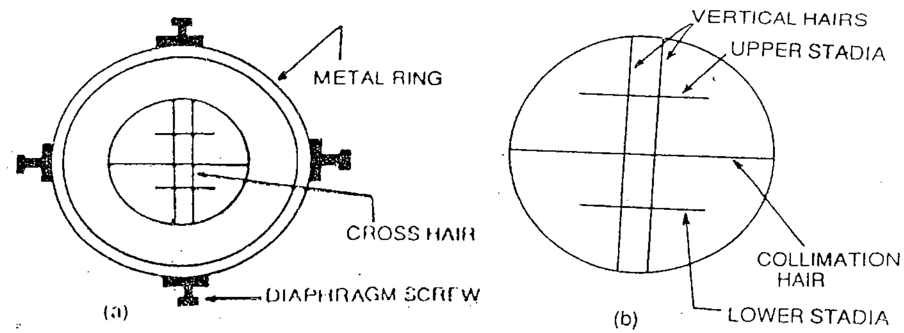


Fig. 5.8

1. Two vertical hairs meant for maintaining the verticality of the staff,
2. Middle horizontal hair representing the line of collimation,
3. Upper stadia hair and lower stadia hair, both horizontal and short in length. The stadia hairs are meant for determining the horizontal distance between the position of level and that of the staff (Fig. 5.8(b)).

D. Points to be Remembered by Staffman

1. The staff should be made vertical by holding it with both arms while standing behind it.
2. The staff should be held on firm ground.
3. When the telescopic staff is to be extended, care should be taken so that it is perfectly stretched and properly fixed on the spring catcher.
4. The bottom of the staff should be kept clean.

E. Points to be Remembered by Level Man

1. The levelling instrument should be placed at a position suitable for the greatest number of observations to be taken.
2. The instrument should not be too high or too low.
3. The levelling should be done perfectly.
4. The levelling instrument should not be placed on the profile line (i.e. the centre line of the project).
5. The eye-piece should be focussed by holding a sheet of white paper in front of the telescope.
6. The objective should be focussed by pointing the telescope towards the staff.
7. The parallax should be eliminated.
8. The verticality of the staff should be verified by observing the two vertical hairs and by noting the minimum reading on the staff when it is moved along the line of sight.
9. When looked at through the telescope, the staff appears inverted. So, the reading should be taken carefully from the top downwards.

10. After taking the staff reading, the position of the bubble should be verified. If it is disturbed, the reading should be taken again.

F. Procedure for Use of Compass

1. The needle of the compass is released by opening the clamp screw.
2. The telescope is directed to the staff or ranging rod and is bisected perfectly.
3. The needle is allowed to come to rest, i.e. the rotation is completely stopped.
4. The reading is taken by raising or lowering the magnifying glass, this is the magnetic bearing of the line.
5. A small pin fixed on the periphery of the compass box will indicate the bearing of the line. The initial reading of the pin should be noted before observation.

5.4 TEMPORARY ADJUSTMENT OF LEVEL

The adjustments made at every set up of the level before the staff readings are taken are known as temporary adjustments. The following are the different steps to be followed in temporary adjustment.

1. Selection of suitable position A suitable position is selected for setting the level. From this position, it should be possible to take the greatest number of observations without any difficulty. The ground should be fairly level and firm.

2. Fixing level with tripod stand The tripod stand is placed at the required position with its legs well apart, and pressed firmly into the ground.

The level is fixed on the top of the tripod stand according to the fixing arrangement provided for that particular level. It should be remembered that the level is not to be set up at any station or point along the alignment.

3. Approximate levelling by legs of tripod stand The foot screws are brought to the centre of their run. Two legs of the tripod stand are firmly fixed into the ground. Then the third leg is moved to the left or right, in or out until the bubble is approximately at the centre of its run.

4. Perfect levelling by foot screws As the longitudinal bubble is on the top of the telescope, the latter is placed parallel to any pair of foot screws (i.e. first position) and the bubble is brought to the centre by turning the foot screws equally either both inwards or both outwards. The telescope is then turned through 90° (i.e. second position) and brought over the third foot screw, and the bubble is brought to the centre by turning this foot screw clockwise or anticlockwise (Fig. 5.9). The telescope is again brought to its original position (the first position) and the bubble is brought to the centre. The process is repeated several times until the bubble remains in the central position in the first as well as the second position. Then the telescope is turned through 180°. If the bubble still remains in the central position, the temporary adjustment is perfect and so is the permanent adjustment. But if the bubble is deflected from its central position, the permanent adjustment is not perfect and needs to be modified. (Permanent adjustment is described in Sec. 5.13).

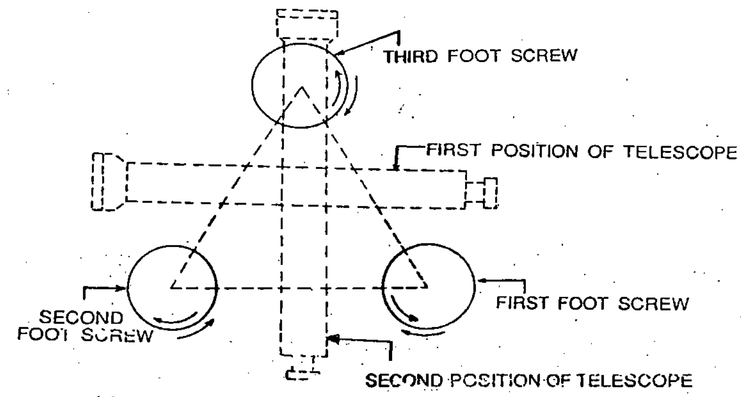


Fig. 5.9

5. Focussing the eye-piece A piece of white paper is held in front of the object glass and the eye-piece is moved in or out by turning it clockwise or anticlockwise until the cross-hairs can be seen clearly.

6. Focussing the object glass The telescope is directed towards the levelling staff. Looking through the eye-piece, the focussing screw is turned clockwise or anticlockwise until the graduation of the staff is distinctly visible and the parallax is eliminated. To eliminate the parallax, the eye is moved up and down to verify whether the graduation of the staff remains fixed relative to the cross-hairs.

7. Taking the staff readings Finally, the levelling of the instrument is verified by turning the telescope in any direction. When the bubbles (the longitudinal bubble and cross bubbles) remain in the central position for any direction of the telescope, the staff readings are taken.

5.5 TYPES OF LEVELLING OPERATION

1. Simple levelling When the difference of level between two points is determined by setting the levelling instrument midway between the points, the process is called simple levelling.

Suppose A and B are two points whose difference of level is to be determined. The level is set up at O, exactly midway between A and B. After proper temporary adjustment, the staff readings on A and B are taken. The difference of these readings gives the difference of level between A and B (Fig. 5.10).

2. Differential levelling Differential levelling is adopted when: (i) the points are a great distance apart, (ii) the difference of elevation between the points is large, (iii) there are obstacles between the points.

This method is also known as compound levelling or continuous levelling. In this method, the level is set up at several suitable positions and staff readings are taken at all of these.

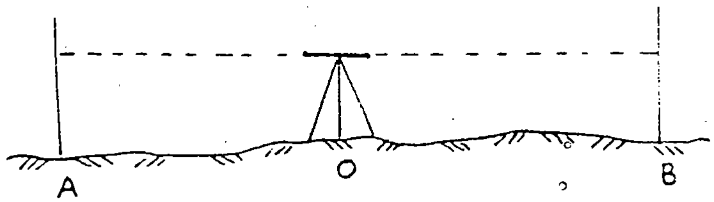


Fig. 5.10

Consider Fig. 5.11. Suppose it is required to know the difference of level between A and B. The level is set up at points O_1, O_2, O_3, etc. After temporary adjustments, staff readings are taken at every set up. The points C_1, C_2 and C_3 are known as change points. Then the difference of level between A and B is found out. If the difference is positive, A is lower than B. If it is negative, A is higher than B.

Knowing the RL of A, that of B can be calculated.

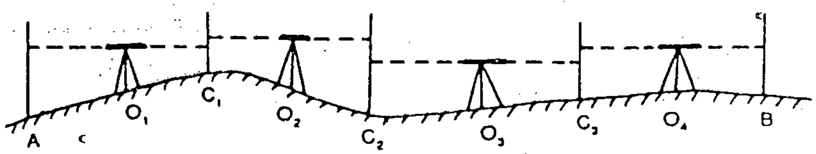


Fig. 5.11

3. Fly levelling When differential levelling is done in order to connect a bench-mark to the starting point of the alignment of any project, it is called fly levelling. Fly levelling is also done to connect the BM to any intermediate point of the alignment for checking the accuracy of the work. In such levelling, only the back-sight and fore-sight readings are taken at every set up of the level and no distances are measured along the direction of levelling (Fig. 5.12). The level should be set up just midway between the BS and the FS.

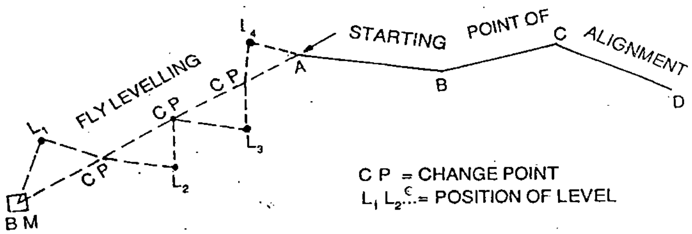


Fig. 5.12

4. Longitudinal or profile levelling The operation of taking levels along the centre line of any alignment (road, railway, etc.) at regular intervals is known as longitudinal levelling. In this operation, the backsight, intermediate sight and foresight readings are taken at regular intervals, at every set up of the instrument. The chainages of

the points are noted in the level book. This operation is undertaken in order to determine the undulations of the ground surface along the profile line (Fig. 5.13).

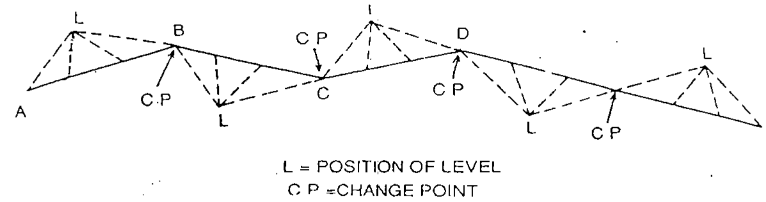


Fig. 5.13

5. Cross-sectional levelling The operation of taking levels transverse to the direction of longitudinal levelling, is known as cross-sectional levelling. The cross-sections are taken at regular intervals (such as 20 m, 40 m, 50 m, etc.) along the alignment. Cross-sectional levelling is done in order to know the nature of the ground across the centre line of any alignment (Fig. 5.14).

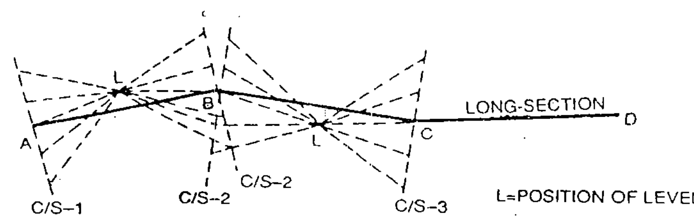


Fig. 5.14

6. Check levelling The fly levelling done at the end of day's work to connect the finishing point with the starting point on that particular day is known as check levelling. It is undertaken in order to check the accuracy of the day's work (Fig. 5.15).

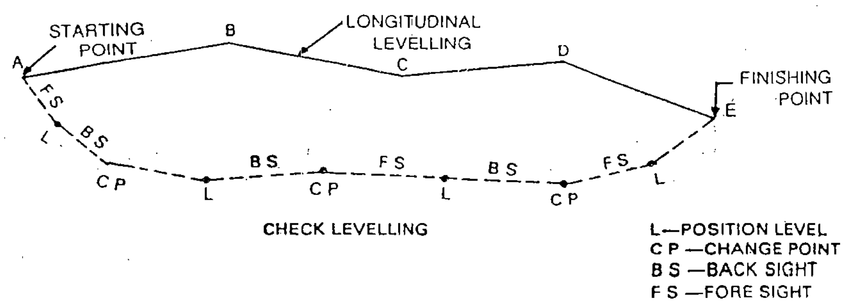


Fig. 5.15

5.6 PRINCIPLE OF EQUALISING BACKSIGHT AND FORESIGHT DISTANCES

In levelling, the line of collimation should be horizontal when the staff readings are taken. Again, the fundamental relation is that the line of collimation should be exactly parallel to the axis of the bubble. So, when the bubble is at the centre of its run, the line of collimation is just horizontal. But sometimes the permanent adjustment of level may be disturbed and the line of collimation may not be parallel to the axis of the bubble. In such a case, due to the inclination of the line of collimation, errors in levelling are likely to occur. But it is found that if the backsight and foresight distances are kept equal, then the error due to the inclination of the collimation line is automatically eliminated, as illustrated below.

Case I—When the line of collimation is inclined upwards Let A and B be two points whose true difference of level is required. The level is set up at O, exactly midway between A and B (Fig. 5.16).

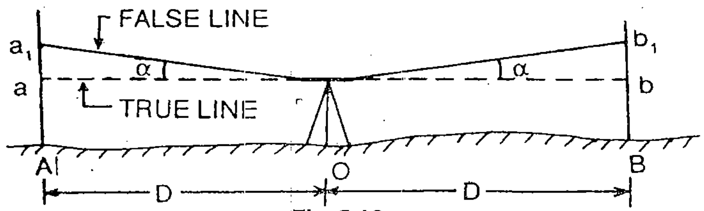


Fig. 5.16

Let α = angle of inclination of collimation line

Aa = true reading

Aa_1 = observed staff reading on A

$\therefore$ Error $= Aa_1 - Aa = aa_1 = D\tan\alpha$

So True reading $Aa = Aa_1 - aa_1 = Aa_1 - D\tan\alpha$ (1)

Similarly, Bb = true reading

Bb_1 = observed staff reading on B

$\therefore$ Error $= Bb_1 - Bb = bb_1 = D\tan\alpha$

So True reading $Bb = Bb_1 - bb_1 = Bb_1 - D\tan\alpha$ (2)

From (1) and (2),

True difference of level between A and B $= Aa - Bb$ (fall from B to A)

$= Aa_1 - D\tan\alpha - Bb_1 + D\tan\alpha$

$= Aa_1 - Bb_1$

Thus, it is seen that the error due to inclination of the collimation line is completely eliminated and the apparent difference is equal to the true difference.

Case II—When the line of collimation is inclined downwards The staff readings on A and B are taken after setting the level at O. Suppose the readings are a_2 and b_2 (Fig. 5.17).

Here, Aa = true staff reading

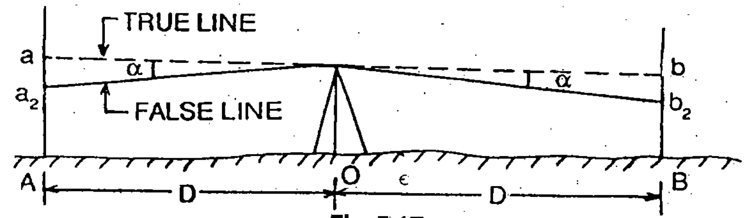


Fig. 5.17

Aa_2 = observed staff reading on A

$\therefore$ Error $= Aa - Aa_2 = aa_2 = D\tan\alpha$

So, True reading $Aa = Aa_2 + aa_2 = Aa_2 + D\tan\alpha$ (1)

Similarly, Bb = true reading

Bb_2 = observed staff reading on B

$\therefore$ Error $= Bb - Bb_2 = bb_2 = D\tan\alpha$

So True reading $Bb = Bb_2 + bb_2 = Bb_2 + D\tan\alpha$ (2)

From (1) and (2),

True difference to level between A and B $= Aa - Bb$ (fall from B to A)

$= Aa_2 + D\tan\alpha - Bb_2 - D\tan\alpha$

$= Aa_2 - Bb_2$

Thus, it is seen that the error due to inclination of the collimation line is completely eliminated.

Note: So, always remember that the level should be placed exactly midway between backsight and foresight in order to eliminate any error.

5.7 CORRECTIONS TO BE APPLIED

1. Curvature correction For long sights, the curvature of the earth affects staff readings. The line of sight is horizontal, but the level line is curved and parallel to the mean spheroidal surface of the earth (Fig. 5.18).

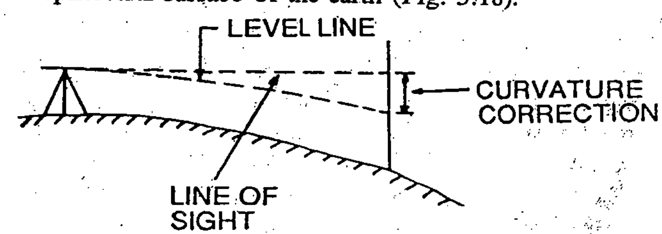


Fig. 5.18

The vertical distance between the line of sight and the level line at a particular place is called the curvature correction. Due to curvature, objects appear lower than they really are.

Curvature correction is always subtractive (i.e. negative)

The formula for curvature correction is derived as follows.

Let AB = D = horizontal distance in kilometres.
BD = Cc = curvature correction
DC = AC = R = radius of earth
DD′ = diameter, considered as 12,742 km

From right-angled triangle ABC (Fig. 5.19).

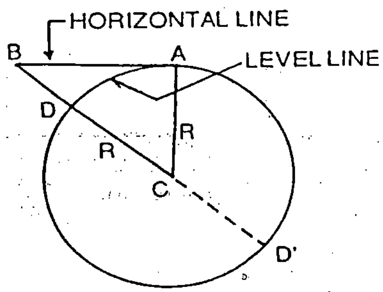


Fig. 5.19

$$BC^2 = AC^2 + AB^2$$

$$(R + C_c)^2 = R^2 + D^2$$

or $$R^2 + 2RC_c + C_c^2 = R^2 + D^2$$

or $$C_c \times 2R = D^2$$

Curvature correction $$C_c = \frac{D^2}{2R}$$

(C_c^2 is neglected as it is very small in comparison to the diameter of the earth.)

$$C_c = \frac{D^2 \times 1{,}000}{12{,}742} = 0.0785\,D^2 \text{ m} \qquad \text{(negative)}$$

Hence, True staff reading = observed staff reading – curvature correction

2. Refraction correction Rays of light are refracted when they pass through layers of air of varying density. So, when long sights are taken, the line of sight is refracted towards the surface of the earth in a curved path. The radius of this curve is seven times that of the earth under normal atmospheric conditions. Due to the effect of refraction, objects appear higher than they really are. But the effect of curvature varies with atmospheric conditions.

However, on an average, the refraction correction is taken as one-seventh of the curvature correction.

$$C_r = \frac{1}{7} \times \frac{D^2}{2R}$$

Refraction correction, $$C_r = \frac{1}{7} \times 0.0785\,D^2 = 0.0112\,D^2 \text{ m} \qquad \text{(positive)}$$

Refraction correction is always additive (i.e. positive).

True staff reading = observed staff reading + refraction correction

3. Combined correction The combined effect of curvature and refraction is as follows:

$$\text{Combined correction} = \text{curvature correction} + \text{refraction correction}$$
$$= -\,0.0785\,D^2 + 0.0112\,D^2$$
$$= -\,0.0673\,D^2 \text{ m}$$

So, combined correction is always subtractive (i.e. negative).

True staff reading = observed staff reading – combined correction

Note: Combined correction may also be expressed as

$$\frac{D^2}{2R} - \frac{1}{7} \times \frac{D^2}{2R} = \frac{6}{7} \cdot \frac{D^2}{2R} \qquad \text{(negative)}$$

4. Visible horizon distance

Let AB = D = visible horizon distance in kilometres (Fig. 5.20)
h = height of the point above mean sea level, in metres

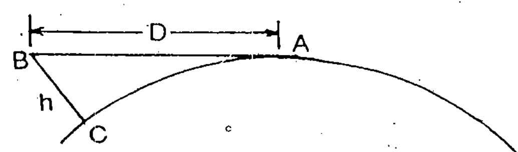


Fig. 5.20

Considering curvature and refraction corrections,

$$h = 0.0673\,D^2$$

$$D = \sqrt{\frac{h}{0.0673}}$$

5. Dip of Horizon

AB = D = tangent to the earth at A
BD = horizontal line perpendicular to OB
θ = dip of horizon

The angle between the horizontal line and the tangent line is known as the dip of the horizon. It is equal to the angle subtended by the arc CA at the centre of the earth (Fig. 5.21).

$$\text{Dip } \theta = \frac{\text{arc CA}}{\text{radius of the earth}}, \text{ in radians}$$

$$\therefore \quad \theta = \frac{D}{R} \text{ in radians} \qquad \text{(Taking CA approx. equal to AB)}$$

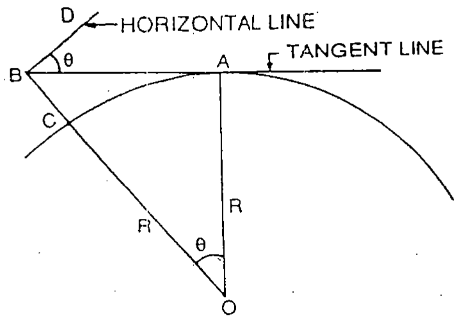


Fig. 5.21

Here, D and R must be expressed in the same units.

6. Sensitiveness of the bubble The term sensitiveness in the context of a bubble means the effect caused by the deviation of the bubble per division of the graduation of the bubble tube.

Sensitiveness is expressed in terms of the radius of curvature of the upper surface of the tube or by an angle through which the axis is tilted for the deflection of one division of the graduation.

Determining sensitiveness Consider Fig. 5.22. Suppose the level was set up at 0 at a distance D from the staff at P. The staff reading is taken with the bubble at the extreme right end (i.e. at E). Say it is PA. Another staff reading is taken with the bubble at the extreme left end (i.e. at E_1). Let it be PB.

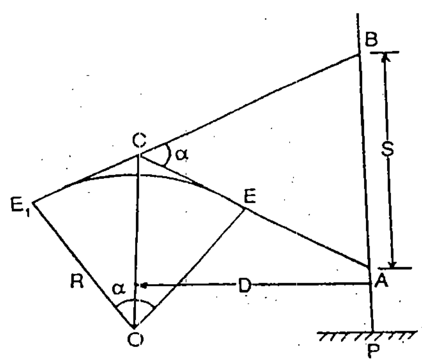


Fig. 5.22

Let D = distance between the level and staff,
S = intercept between the upper and lower sights,
n = number of divisions through which the bubble is deflected,
R = radius of curvature of the tube,
α = angle subtended by arc EE_1, and
d = length of one division of the graduation, expressed in the same units as D and S.

Movement of centre of bubble = $EE_1 = nd$.

Triangles OEE_1 and ACB are similar.

Here $R\alpha$ = arc EE_1

or $$\alpha = \frac{EE_1}{R} = \frac{nd}{R} \quad (1)$$

(as arc $EE_1 \approx$ chord EE_1)

Again $$\frac{EE_1}{R} = \frac{S}{D}$$ (height of ΔOEE_1 may be considered as R)

or $$\frac{nd}{R} = \frac{S}{D} \quad (2)$$

From (1) and (2), $$\alpha = \frac{nd}{R} = \frac{S}{D} \quad (3)$$

$$\therefore \quad R = \frac{nd \times D}{S}$$

Let α' = angular value for one division in radians

$$\alpha' = \frac{\alpha}{n} = \frac{S}{D} \times \frac{1}{n} \text{ radians}$$

or $$\alpha' = \frac{S}{Dn} \times 206,265 \text{ secs} \quad (1 \text{ radian} = 206,265 \text{ secs})$$

5.8 PROBLEMS ON CORRECTIONS AND SENSITIVENESS

Problem 1 A level is set up at a point 150 m from A and 100 m from B; the observed staff readings at A and B are 2.525 and 1.755 respectively. Find the true difference of level between A and B.

Solution Combined correction for curvature and refraction to staff reading at

$$A = 0.0673 \times D^2 = 0.0673 \times \left(\frac{150}{1,000}\right)^2 = 0.0015 \text{ m}$$

$$\text{Correct reading on A} = 2.5250 - 0.0015 = 2.5235 \text{ m} \quad (1)$$

Combined correction for curvature and refraction to staff reading at

$$B = 0.0673 \times \left(\frac{100}{1,000}\right)^2 = 0.000673 \text{ m} = 0.0007 \text{ m} \quad \text{(say)}$$

Correct reading at B = 1.7550 − 0.0007 = 1.7543 m

True difference of level between A and B = 2.5235 − 1.7543 = 0.7692 m (fall from B to A)

Problem 2 A lamp at the top of a lighthouse is visible just above the horizon from a station at sea level. The distance of the lamp from the station is 30 km. Find the height of the lighthouse.

Solution The lamp is visible above the horizon due to the combined effect of curvature and refraction.

We know that $h = 0.0673\ D^2$ (here AL = D = 30 km)

$= 0.0673 \times (30)^2 = 60.57$ m

Hence the height of the lighthouse is 60.57 m.

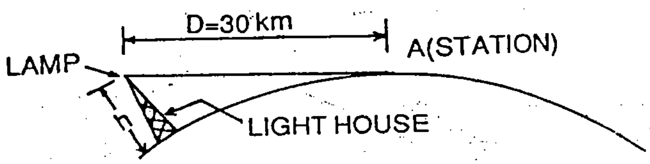


Fig. P-5.1

Problem 3 What is the visible horizon distance from a tower 50 m high? What is the dip of the horizon, assuming the radius of the earth to be 6,370 km?

Solution From the relation,

$h = 0.0673\ D^2$ where D = visible horizon distance, and

$D = \sqrt{\dfrac{h}{0.0673}}$ h = height of tower = 50 m

$D = \sqrt{\dfrac{50}{0.0673}} = 27.26$ km

Dip of horizon $= \dfrac{D}{R}$ radians $= \dfrac{27.26}{6{,}370}$ radians

$= \dfrac{27.26}{6{,}370} \times \dfrac{180 \times 60}{\pi}$ mins = 14.71 mins

Problem 4 A man on the deck of a ship oberves a luminous object, which is 50 m above sea level. If the man's eye-level is 10 m above sea level, find the distance between him and the object.

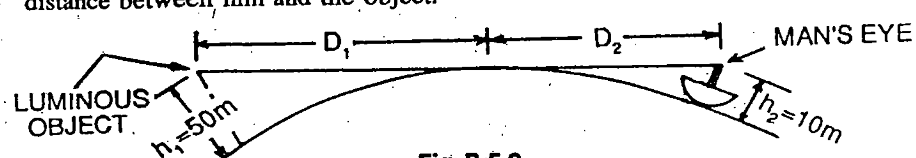


Fig. P-5.2

Solution From the figure, it is clear that the required distance will be equal to $D_1 + D_2$.

Now $D_1 = \sqrt{\dfrac{50}{0.0673}} = 27.26$ km

$D_2 = \sqrt{\dfrac{10}{0.0673}} = 12.19$ km

Hence, distance between man and object = $D_1 + D_2$

= 27.26 + 12.19 = 39.45 km

Problem 5 A man at a position 10 m above sea level observes the peak of a hill. The distance between the man and the hill is 80 km. Find the height of the hill.

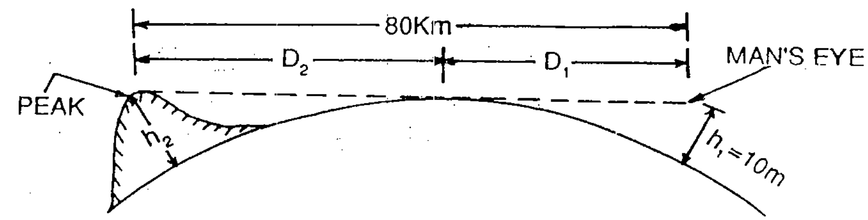


Fig. P-5.3

Solution From the relation

$h_1 = 0.0673\ D_1^2$ Here, $D_1 + D_2 = 80$ km

$D_1 = \sqrt{\dfrac{10}{0.0673}} = 12.19$ km $h_1 = 10$ m, height of man's eye

$D_2 = 80 - 12.19 = 67.81$ km h_2 = height of the hill

$h_2 = 0.0673 \times D_2^2 = 0.0673 \times (67.81)^2 = 309.46$ m

Hence the height of the hill is 309.46 m.

Problem 6 The line of sight from two stations A and B just grazes the sea level. If the height of A and B above sea level are 100 and 150 m respectively, find the distance AB (diameter of earth = 12,880 km).

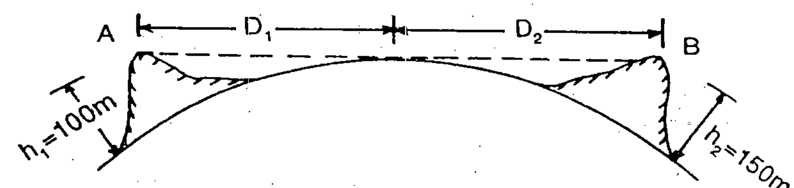


Fig. P-5.4

Solution We know that combined correction

$= \dfrac{6}{7} \times \dfrac{1{,}000\ D^2}{2R}$

$= \dfrac{6}{7} \times \dfrac{1{,}000\ D^2}{12{,}880}$ (R = radius of the earth)

$= 0.0665\ D^2$ ($2R$ = diameter = 12,880 km) so $h_1 = 0.0665\,D_1^2$

$$D_1 = \sqrt{\frac{h_1}{0.0665}} = \sqrt{\frac{100}{0.0665}} = 38.78 \text{ km} \qquad (h_1 = 100 \text{ m})$$

and $h_2 = 0.0665\,D_2^2$

$$D_2 = \sqrt{\frac{150}{0.0665}} = 47.49 \text{ km} \qquad (h_2 = 150 \text{ m})$$

$$\text{Distance AB} = D_1 + D_2 = 38.78 + 47.49 = 86.27 \text{ km}$$

Problem 7 When the bubble is at the centre, the reading on the staff, 100 m from the level, is 2.550 m. The bubble is then deviated by five divisions and the staff reading is 2.500 m. If the length of one division of the bubble is 2 mm, calculate the radius of curvature of the bubble tube and the angular value of one division of the bubble.

Solution Here, staff intercept, $S = 2.550 - 2.500$
$= 0.050$ m

$n = 5$ divs (deviation)

$d = 2$ mm $= 0.002$ m (length of one division)

$D = 100$ m (distance between level and staff)

From the relation

$$R = \frac{nD}{S} = \frac{5 \times 0.002 \times 100}{0.050} = 20 \text{ m}$$

The sensitiveness of the bubble α' is

$$\alpha' = \frac{S}{nD} \times 206,265 \text{ sec} = \frac{0.05 \times 206,265}{5 \times 100}$$

$$= 20.62 \text{ secs}$$

So, the radius of curvature is 20 m and the angular value of one division is 20.62 secs.

Problem 8 A bubble tube of a level has a sensitiveness of 20″ per 2 mm division. Find the error in the reading on the staff held at a distance of 100 m from the level when the bubble is deflected by two divisions from the centre.

Solution We know that the angular value of one division

$$\alpha' = \frac{S}{nD} \times 206,265 \text{ sec}$$

Here $\alpha' = 20$ sec,

$S = ?$ (staff intercept),

$n = 2$ divisions (deflection), and

D = distance of the staff from level = 100 m.

$$20 = \frac{S}{2 \times 100} \times 206,265$$

$$S = \frac{20 \times 2 \times 100}{206,265} = 0.019 \text{ m}$$

So, the error is 0.019 m.

5.9 RECIPROCAL LEVELLING

We have already found by the principle of equalising backsight and foresight distances that if the level is placed exactly midway between two points and staff readings are taken to determine the difference of level, then the errors (due to inclined collimation line, curvature and refraction) are automatically eliminated. But in the case of a river or valley, it is not possible to set up the level midway between two points on opposite banks. In such cases, the method of reciprocal levelling is adopted, which involves reciprocal observations from both banks of the river or valley.

In reciprocal levelling, the level is set up on both banks of the river or valley and two sets of staff readings are taken by holding the staff on both banks. In this case, it is found that the errors are completely eliminated and the true difference of level is equal to the mean of the two apparent differences of level.

The principle is explained as follows.

Procedure 1. Suppose A and B are two points on the opposite banks of a river. The level is set up very near A and after proper temporary adjustment, staff readings are taken at A and B. Suppose the readings are a_1 and b_1 (Fig. 5.23(a)).

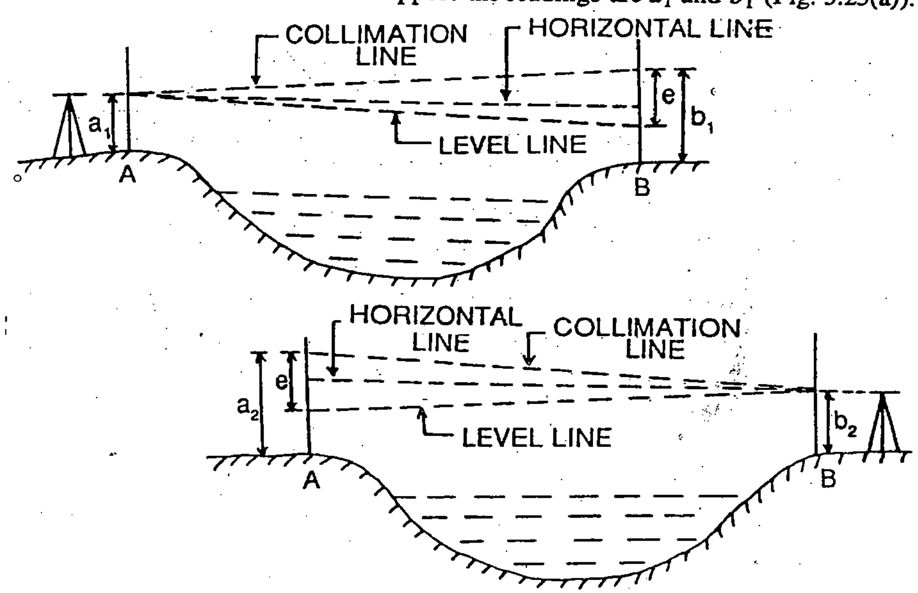


Fig. 5.23 (a) & (b)

2. The level is shifted and set up very near B and after proper adjustment, staff readings are taken at A and B. Suppose the readings are a_2 and b_2 (Fig. 5.23(b)).

Let h = true difference of level between A and B
e = combined error due to curvature, refraction and collimation (The error may be positive or negative, here the error is assumed positive)

In the first case,

Correct staff reading at A = a_1 (as the level is very near A)
Correct staff reading at B = $b_1 - e$

True difference of level between A and B,

$$h = a_1 - (b_1 - e) \quad \text{(fall from B to A)} \qquad (1)$$

In the second case,

Correct staff reading at B = b_2 (as level is near B)
Correct staff reading at A = $a_2 - e$

So, true difference of level,

$$h = (a_2 - e) - b_2 \qquad (2)$$

From (1) and (2),

$$2h = a_1 - (b_1 - e) + (a_2 - e) - b_2$$
$$= a_1 - b_1 + e + a_2 - e - b_2 = (a_1 - b_1) + (a_2 - b_2)$$
$$h = \frac{(a_1 - b_1) + (a_2 - b_2)}{2}$$

It may be observed that the error is eliminated and that the true difference is equal to the mean of the two apparent differences of level between A and B.

5.10 PROBLEMS ON RECIPROCAL LEVELLING

Problem 1 In an operation involving reciprocal levelling, two points A and B are taken on opposite banks of a river. When the level was set up near A, the staff readings on A and B were 2.245 and 3.375 respectively. When the level was set up near B, the respective staff readings were 1.955 and 3.055. Find the true difference of level between A and B. What is the RL of B, if that of A is 125.550?

Solution In the first setting,
Apparent difference of level between

A and B = 3.375 – 2.245 = 1.130 m (Fall from A to B)

In the second setting,
Apparent difference of level between A and B = 3.055 – 1.955 = 1.100 m

$$\text{True difference of level} = \frac{1.130 + 1.100}{2} = 1.115 \text{ m}$$

RL of B = 125.550 – 1.115 = 124.435 m

Problem 2 The following records refer to an operation involving reciprocal levelling.

Instrument at	Staff reading on		Remarks
	A	B	
A	1.155	2.595	Distance AB = 500 m
B	0.985	2.415	RL of A = 525.500

Find:

(a) The true RL of B,
(b) The combined correction for curvature and refraction,
(c) The collimation error, and
(d) Whether the line of collimation is inclined upwards or downwards.

Solution

(a) True difference of level between A and B

$$= \frac{(2.595 - 1.155) + (2.415 - 0.985)}{2}$$
$$= 1.435 \text{ m} \quad \text{(fall from A to B)} \qquad (1)$$

RL of B = 525.500 – 1.435 = 524.065 m

(b) Combined correction for 500 m = $0.0673 \times (0.5)^2 = 0.0168$ m (negative)

(c) Let us assume that the line of collimation is inclined upwards.
Let, Collimation error in 500 m = e (positive, as it is inclined upwards)

(*Note*: When the error is positive, correction will be negative and vice versa.)

When the instrument is at A,
Correct staff reading at A = 1.155 m (as level is near A)
Correct staff reading at B = (2.595 – 0.0168 – e)

True difference of level between A and B

$$= (2.595 - 0.0168 - e) - 1.155$$
$$= 1.4232 - e \qquad (2)$$

From (1) and (2),

$$1.4232 - e = 1.4350 \quad e = -0.0118$$

$$\text{Collimation error per 100 m} = -\frac{0.0118 \times 100}{500} = -0.0023 \text{ m}$$

(d) The collimation error was assumed positive, but the result is negative. So the assumption is wrong. The line of collimation is actually inclined downwards.

Problem 3 In testing a dumpy level, the following records were noted while undertaking reciprocal levelling:

Instrument at	Reading at	
	A	B
A B	1.725 1.560	1.370 1.235

Is the line of collimation in adjustment? What should be the correct staff reading at A, during the second set up to make the line of collimation truly horizontal? Find the amount of collimation error also.

Solution When the instrument is at A

Apparent difference of level = 1.725 – 1.370 = 0.355 m

When the instrument is at B,

Apparent difference of level = 1.560 – 1.235 = 0.325 m

Since the two apparent differences are not equal, the line of collimation is not in adjustment.

True difference of level

$$\text{between A and B} = \frac{0.355 + 0.325}{2} = 0.340 \text{ m} \qquad \text{(fall from B to A)}$$

In second set up

Correct reading at B = 1.235 m (as the level is near B)
Correct staff reading at A = 1.235 + 0.340 = 1.575 m

But the observed staff reading (1.560 m) at A is less than the correct reading (1.575 m). So, the line of collimation is inclined downwards.

Amount of collimation error = 1.560 – 1.575 = – 0.015 m

Problem 4 The following observations were made during the testing of a dumpy level.

Instrument at	Staff reading at		Remark
	A	B	
A B	1.725 2.145	2.245 3.045	RL of A = 450.000 m

Distance between A and B = 200 m

Is the instrument in adjustment? To what reading should the line of collimation be adjusted when the instrument is at B? Find the RL of B.

Solution *When the instrument is at A,*
Apparent difference of level between A and B = 2.245 – 1.725 = 0.520 m

When the instrument is at B,

Apparent difference of level = 3.045 – 2.145 = 0.900 m

Since the two apparent differences are not equal, the line of collimation is not in adjustment.

$$\text{True difference of level} = \frac{0.520 + 0.900}{2} = 0.710 \text{ m} \qquad \text{(fall from A to B)}$$

When the instrument is at B,
Correct staff reading at B = 3.045 m (as the level was near B)
Correct staff reading at A = 3.045 – 0710 = 2.335 m

But the observed staff reading at A (2.145 m) is less than the true staff reading (2.335 m), so the line of collimation is inclined downwards.

Collimation error = 2.145 – 2.335 = – 0.190 m

So, a correction of + 0.190 m should be applied at A.

RL of B = 450.000 – 0.710 = 449.290 m

5.11 METHODS OF CALCULATION OF REDUCED LEVEL

The following are the two systems of calculating reduced level:

1. The collimation system or height of instrument system (HI)
2. The rise-and-fall system.

1. The collimation system The reduced level of the line of collimation is said to be the height of the instrument. In this system, the height of the line of collimation is found out by adding the backsight reading to the RL of the BM on which the BS is taken. Then the RL of the intermediate points and the change point are obtained by subtracting the respective staff readings from the height of the instrument (HI).

The level is then shifted for the next se tup and again the height of the line of collimation is obtained by adding the backsight reading to the RL of the change point (which was calculated in the first set up).

So, the height of the instrument is different in different setups of the level. Two adjacent planes of collimation are correlated at the change point by an FS reading from one setting and a BS reading from the next setting.

It should be remembered that, in this system, the RLs of unknown points are to be found out by deducting the staff readings from the RL of the height of the instrument.

Consider Fig. 5.24.

(a) RL of HI in 1st setting = 100.000 + 1.255 = 101.255
RL of A = 101.255 – 1.750 = 99.505
RL of B = 101.255 – 2.150 = 99.105
(b) RL of HI in 2nd setting = 99.105 + 2.750 = 101.855
RL of C = 101.855 – 1.950 = 99.905

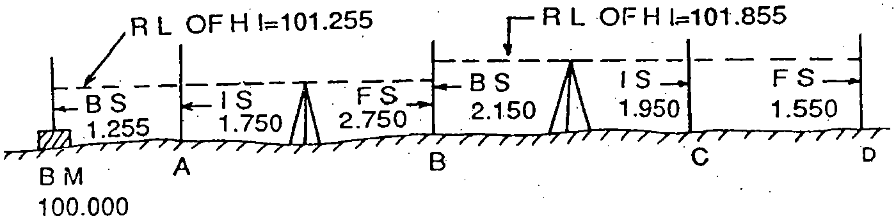


Fig. 5.24

RL of D = 101.855 − 1.550 = 100.305 and so on.

Arithmetical check: Σ BS − Σ FS = Last RL − 1st RL

The difference between the sum of backsights and that of foresights must be equal to the difference between the last RL and the first RL. This check verifies the calculation of the RL of the HI and that of the change point. There is no check on the RLs of the intermediate points.

2. The rise-and-fall system In this system, the difference of level between two consecutive points is determined by comparing each forward staff reading with the staff reading at the immediately preceding point.

If the forward staff reading is smaller than the immediately preceding staff reading, a rise is said to have occurred. The rise is added to the RL of the preceding point to get the RL of the forward point.

If the forward staff reading is greater than the immediately preceding staff reading, it means there has been a fall. The fall is subtracted from the RL of preceding point to get the RL of the forward point.

Consider Fig. 5.25.

Point A (with respect to BM) = 0.75 − 1.25 = − 0.50 (fall)
Point B (with respect to A) = 1.25 − 2.75 = − 1.50 (fall)
Point C (with respect to B) = 2.75 − 1.50 = + 1.25 (rise)
Point D (with respect to C) = 1.50 − 1.75 = − 0.25 (fall)

RL of BM = 100.00
RL of A = 100.00 − 0.50 = 99.50
RL of B = 99.50 − 1.50 = 98.00
RL of C = 98.00 + 1.25 = 99.25
RL of D = 99.25 − 0.25 = 99.00

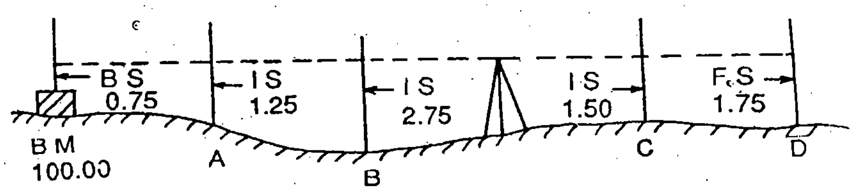


Fig. 5.25

Arithmetical check: ΣBS − ΣFS = Σrise − Σfall = last RL − 1st RL

In this method, the difference between the sum of BSs and that of FSs, the difference between the sum of rises and that of falls and the difference between the last RL, and the first RL must be equal.

Note: The arithmetical check is meant only for the accuracy of calculation to be verified. It does not verify the accuracy of field work. There is a complete check on the RLs of intermediate points in the rise-and-fall system.

Comparison of the two systems

Collimation System	Rise-and-Fall System
1. It is rapid as it involves few calculation.	It is laborious, involving several calculations.
2. There is no check on the RL of intermediate points.	There is a check on the RL of intermediate points.
3. Errors in intermediate RLs cannot be detected.	Errors in intermediate RLs can be detected as all the points are correlated.
4. There are two checks on the accuracy of RL calculation.	There are three checks on the accuracy of RL calculation.
5. This system is suitable for longitudinal levelling where there are a number of intermediate sights.	This system is suitable for fly levelling where there are no intermediate sights.

Considering the above points, the rise-and-fall system is always preferred as there is no possibility of error in the calculation of RLs in the intermediate points.

5.12 POINTS TO BE REMEMBERED WHILE ENTERING THE LEVEL BOOK

1. The first reading of any set up is entered in the BS column, the last reading in the FS column and the other readings in the IS column.
2. A page always starts with a BS reading and finishes with an FS reading.
3. If a page finishes with an IS reading, the reading is entered in the IS and FS columns on that page and brought forward to the next page by entering it in the BS and IS columns.
4. The FS and BS of any change point are entered in the same horizontal line.
5. The RL of the line of collimation is entered in the same horizontal line in which the corresponding BS was entered.
6. Important note, bench-marks and change points should be clearly described in the remark column.

Example The following consecutive readings were taken with a dumpy level along a chain line at a common interval of 15 m. The first reading was at a chainage of 165 m where the RL is 98.085. The instrument was shifted after the fourth and ninth readings.

3.150, 2.245, 1.125, 0.860, 3.125, 2.760, 1.835, 1.470, 1.965, 1.225, 2.390, and 3.035 m.

Mark rules on a page of your notebook in the form of a level book page and enter on it the above readings and find the RL of all the points by:

1. The collimation system, and
2. The rise-and-fall system.

Apply the usual checks.

1. By the collimation system:

Station point	Chainage	BS	IS	FS	RL of collimation line (HI)	RL	Remark
1	165	3.150			101.235	98.085	
2	180		2.245			98.990	
3	195		1.125			100.110	
4	210	3.125		0.860	103.500	100.375	changed point
5	225		2.760			100.740	
6	240		1.835			101.665	
7	255		1.470			102.030	
8	270	1.225		1.965	102.760	101.535	Change point
9	285		2.390			100.370	
10	300			3.035		99.725	
Total =		7.500		5.860			

Arithmetical check:

$$\Sigma BS - \Sigma FS = 7.500 - 5.860 = +1.640$$
$$\text{Last RL} - \text{1st RL} = 99.725 - 99.085 = +1.640$$

2. By the rise-and-fall system:

Station point	Chainage	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
1	165	3.150					98.085	
2	180		2.245		0.905		98.990	
3	195		1.125		1.120		100.110	
4	210	3.125		0.860	0.265		100.375	changed point
5	225		2.760		0.365		100.740	
6	240		1.835		0.925		101.665	
7	255		1.470		0.365		102.030	
8	270	1.225		1.965		0.495	101.535	changed point
9	285		2.390			1.165	100.370	
10	300			3.035		0.645	99.725	
Total =		7.500		5.860	3.945	2.305		

Arithmetical check:

$$\Sigma BS - \Sigma FS = 7.500 - 5.860 = +1.640$$
$$\Sigma \text{Rise} - \Sigma \text{fall} = 3.945 - 2.305 = +1.640$$
$$\text{Last RL} - \text{1st RL} = 99.725 - 98.085 = +1.640$$

5.13 PROBLEMS ON REDUCTION OF LEVELS

Problem 1 The following consecutive readings were taken with a levelling instrument at intervals of 20 m.

2.375, 1.730, 0.615, 3.450, 2.835, 2.070, 1.835, 0.985, 0.435, 1.630, 2.255 and 3.630 m.

The instrument was shifted after the fourth and eighth readings. The last reading was taken on a BM of RL 110.200 m. Find the RLs of all the points.

Solution

Station point	Chainage	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
1	0	2.375					112.620	
2	20		1.730		0.645		113.265	
3	40		0.615		1.115		114.380	
4	60	2.835		3.450		2.835	111.545	Change point
5	80		2.070		0.765		112.310	
6	100		1.835		0.235		112.545	
7	120	0.435		0.985	0.850		113.395	Change point
8	140		1.630			1.195	112.200	
9	160		2.255			0.625	111.575	
10	180			3.630		1.375	110.200	On BM
Total =		5.645		8.065	3.610	6.030		

Procedure:

1. First calculate the rise and fall. Then find:

$$\Sigma BS - \Sigma FS = 5.645 - 8.065 = -2.420$$
$$\Sigma \text{Rise} - \Sigma \text{fall} = 3.610 - 6.030 = -2.420$$

2. Then, $\text{Last RL} - \text{1st RL} = -2.420$

or $\text{1st RL} = 110.200 + 2.420 = 112.620$

Substitute this value for the RL of the first point and calculate the other RLs in the usual way.

Problem 2 The following successive readings were taken with a dumpy level along a chain line at common intervals of 20 m. The first reading was taken on a chainage 140 m. The RL of the second change point was 107.215 m. The instrument was shifted after the third and seventh readings. Calculate the RLs, of all the points.

3.150, 2.245, 1.125, 3.860, 2.125, 0.760, 2.235, 0.470, 1.935, 3.225 and 3.890 m.

Solution

Station point	Chainage	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
1	140	3.150					103.565	
2	160		2.245		0.905		104.470	
3	180	3.860		1.125	1.120		105.590	1st CP
4	200		2.125		1.735		107.325	
5	220		0.760		1.365		108.690	
6	240	0.470		2.235		1.475	107.215	2nd CP
7	260		1.935			1.465	105.750	
8	280		3.225			1.290	104.460	
9	300			3.890		0.665	103.795	
Total =		7.480		7.250	5.125	4.895		

Procedure:

1. First calculate the rise and fall as usual. Then find:

$$\Sigma \text{ BS} - \Sigma \text{ FS} = 7.480 - 7.250 = +\ 0.230$$
$$\Sigma \text{ Rise} - \Sigma \text{ Fall} = 5.125 - 4.895 = +\ 0.230$$

2. Now calculate the RL from the second CP to the last point as usual. Then,

$$\text{Last RL} - \text{1st RL} = +\ 0.230$$
$$\text{1st RL} = 103.795 - 0.230 = 103.565$$

Then calculate the RLs of the remaining points.

Problem 3 The following consecutive readings were taken with a level and a 4-metre levelling staff on a continuously sloping ground at common intervals of 30 m:

0.855 (on A), 1.545 m, 2,335, 3.115, 3.825, 0.455, 1.380, 2.055, 2.855, 3.455, 0.585, 1.015, 1.850, 2.755, 3.845 (on B).

The RL of A was 380.500. Make entries in a level book and apply the usual checks. Determine the gradient of AB.

Solution

Station point	Chainage	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
A	0	0.855					380.500	
	30		1.545			0.690	379.810	
	60		2.335			0.790	379.020	
	90		3.115			0.780	378.240	
	120	0.455		3.825		0.710	377.530	Change point
	150		1.380			0.925	376.605	
	180		2.055			0.675	375.930	
	210		2.855			0.800	375.130	
	240	0.585		3.455		0.600	374.530	Change point
	270		1.015			0.430	374.100	
	300		1.850			0.835	373.265	
	330		2.755			0.905	372.360	
B	360			3.845		1.090	371.270	
Total =		1.895		11.125	0	9.230		

Procedure: Observe that the given readings are gradually increasing initially, but that they suddenly decrease after the fifth and tenth readings. This indicates that the instrument was shifted after 5th and 10th readings. Then proceed as usual method.

Check: $\Sigma \text{ BS} - \Sigma \text{ FS} = 1.895 - 11.125 = -\ 9.230$
$\Sigma \text{ Rise} - \Sigma \text{ fall} = 0 - 9.230 = -\ 9.230$
$\text{Last RL} - \text{1st RL} = 371.270 - 380.500 = -\ 9.230$

$$\text{Falling gradient of AB} = \frac{\text{difference of level}}{\text{horizontal distance}}$$
$$= \frac{9.230}{360} = \frac{1}{39} \quad \text{(i.e. 1 in 39)}$$

Problem 4 In fly levelling from a BM of RL 140.605, the following readings were observed:

Backsight—1.545, 2.695, 1.415, 2.925
Foresight—0.575, 1.235, 0.595

From the last position of the instrument, six pegs at 20 m intervals are to be set out on a uniformly rising gradient of 1 in 50, the first peg is to have an RL of 144.000. Find the staff readings and RLs of the pegs.

Solution

Station	Chainage	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
BM		1.545					140.605	On BM
		2.695		0.575	0.970		141.575	
		1.415		1.235	1.460		143.035	
		2.925		0.595	0.820		143.855	
Peg 1	0		2.780		0.145		144.000	
Peg 2	20		2.380		0.400		144.400	
Peg 3	40		1.980		0.400		144.800	
Peg 4	60		1.580		0.400		145.200	
Peg 5	80		1.180		0.400		145.600	
Peg 6	100			0.780	0.400		146.000	
Total =		8.580		3.185	5.395	0		

Procedure:

1. Find the rise and fall from the given BS and FS and calculate the RL up to the last BS.
2. The first peg is at a rise of (144.000 – 143.855) = 0.145 with respect to the last BS point. So, the staff reading on the first peg is 2.925 – 0.145 = 2.780.
3. Again, the rising gradient is 1 in 50.

So $$\text{Rise per 20 m} = \frac{20}{50} = 0.400$$

The staff readings and RLs of the other pegs may be calculated as usual.

Check: Σ BS – Σ FS = Σ rise – Σ fall = last RL – 1st RL
8.580 – 3.185 = 5.395 – 0 = 146.000 – 140.605 + 5.395
= + 5.395 = + 5.395

Problem 5 During a fly levelling operation, the following observations were made:

BS—0.650, 2.155, 1.405, 2.655, 2.435 m
FS—2.455, 1.305, 0.555, 2.405 m

The first backsight was taken on a BM of RL 90.500 m. From the last backsight, it is required to set four pegs each at a distance of 30 m on a falling gradient of 1 in 100. Calculate the RLs of these four pegs.

Solution

Station	Chainage	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
BM		0.650					90.500	On BM
		2.155		2.455		1.805	88.695	
		1.405		1.305	0.850		89.545	
		2.655		0.555	0.850		90.395	
	0	2.435		2.405	0.250		90.645	
Peg 1	30		2.735			0.300	90.345	
Peg 2	60		3.035			0.300	90.045	
Peg 3	90		3.335			0.300	89.745	
Peg 4	120			3.635		0.300	89.445	
Total =		9.300		10.355	1.950	3.005		

Procedure:

1. Find the rise and fall from the given BS and FS as usual.
2. The falling gradient is 1 in 100.

So, $$\text{Fall per 30 m} = \frac{30}{100} = 0.300$$

3. From the last BS the fall of the first peg is 0.300. So, the staff reading for this peg is obtained as 2.735 (2.435 + 0.300).
4. Each peg has a fall of 0.300, and the staff reading and RL can be calculated accordingly in the usual manner.

Check: Σ BS – Σ FS = 9.300 – 10.355 = – 1.055
Σ Rise – Σ Fall = 1.950 – 3.005 = – 1.055
Last RL – 1st RL = 89.445 – 90.500 = – 1.055

Problem 6 The following is the page of a level book, where some readings were missing. Fill in the missing readings and calculate the reduced levels of all the points.

Station	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
1	3.250					2,49.260	BM
2	1.755		?		0.750		CP
3		1.950					
4	?		1.920				CP
5		2.340		1.500			
6		?		1.000			
7	1.850		2.185				CP
8		1.575					
9		?					
10	?		1.895	1.650			CP
11			1.350	0.750			

Solution

Station	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
1	3.250					249.260	BM
2	1.755		4.000		0.750	248.510	CP
3		1.950			0.195	248.315	
4	3.840		1.920	0.030		248.345	CP
5		2.340		1.500		249.845	
6		1.340		1.000		250.845	
7	1.850		2.185		0.845	250.000	CP
8		1.575		0.275		250.275	
9		3.545			1.970	248.305	
10	2.100		1.895	1.650		149.955	CP
11			1.350	0.750		250.705	
Total =	12.795		11.350	5.205	3.760		

Procedure:

Missing data for line 2 = 3.250 + 0.750 = 4.000
Missing data for line 4 = 2.340 + 1.500 = 3.840
Missing data for line 6 = 2.340 – 1.000 = 1.340
Missing data for line 9 = 1.895 + 1.650 = 3.545
Missing data for line 10 = 1.350 + 0.750 = 2.100

Check:

Σ BS – Σ FS = 12.795 – 11.350 = + 1.445
Σ Rise – Σ Fall = 5.205 – 3.460 = + 1.445
Last RL – 1st RL = 250.705 – 249.260 = + 1.445

Problem 7 A page of a level book with some missing readings is reproduced below. Fill in the missing entries along with the necessary arithmetical checks.

Station	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
1	3.650					108.260	
2		×		2.750			
3		2.830					
4		3.640					
5	×		2.420				Ct
6		2.410			1.320		
7		2.320					
8		3.000					
9		– 2.170*					*Negative reading indicates staff held against ceiling
10	×		×		2.750		CP
11			×		1.320		
Total =	6.490						

(Contd.)

Solution

Station	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
1	3.650					108.260	
2		0.900		2.750		1110.10	
3		2.830			1.930	109.080	
4		3.640			0.810	108.270	
5	1.090		2.420	1.220		109.490	CP
6		2.410			1.320	108.170	
7		2.320		0.090		108.260	
8		3.000			0.680	107.580	
9		– 2.170*		5.170		112.750	*Negative reading indicates staff held against ceiling.
10	1.750		0.580		2.750	110.000	CP
11			3.070		1.320	108.680	
Total =	6.490		6.070	9.230	8.810		

Procedure:

Missing data for line 2 = 3.650 – 2.750 = 0.900
Missing data for line 5 = 2.410 – 1.320 = 1.090
Missing data for line 10 (FS) = – 2.170 – (– 2.750) = + 0.580
Missing data for line (10 (BS) = 6.490 – (3.650 + 1.090) = 1.750
Missing data for line 11 = 1.750 + 1.320 = 3.070

Check: Σ BS – Σ FS = 6.490 – 6.070 = + 0.420
Σ Rise – Σ Fall = 9.230 – 8.810 = + 0.420
Last RL – 1st RL = 108.680 – 108.260 = + 0.420

5.14 PROJECT WORK (ROADS, RAILWAYS, ETC.)

Preparation of Road

1. Marking tentative alignment The tentative alignment of the road project is marked on a topographical and contour map of the area. The following points should be considered in this context:

(a) The road should not cross any river obliquely.
(b) It should not pass through depressions, ditches or big ponds.
(c) The road should connect a fair number of villages, towns, and other important places.

During the survey, the tentative alignment may be altered in some places, bearing in mind the following points:

(a) The road should not pass through burial grounds, burning ghats, temples, mosques, churches or any other places of worship.
(b) It should not pass through any valuable permanent structures where the compensation to be paid is likely to be more.

2. Preparation of compass survey map An open traverse is done by prismatic compass along the alignment to understand the nature of the ground and with intersections with other roads, canals, rivers, etc. A map is prepared to a suitable scale for a strip of land, taking about 20 or 40 m on both sides of the centre line of the road. A sample map is shown in Fig. 5.26.

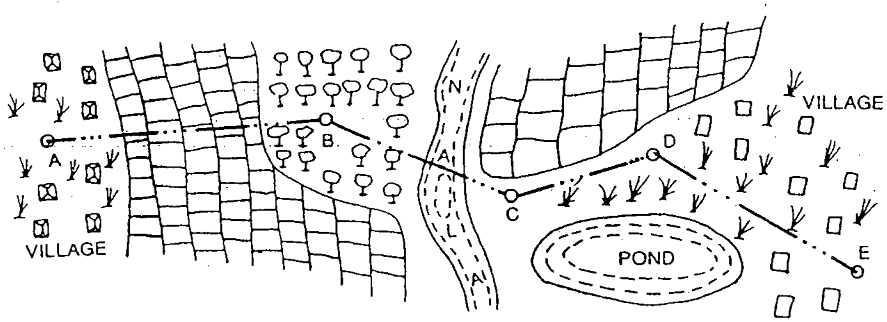


Fig. 5.26

3. Fly levelling The permanent bench-mark may be located far away from the starting point of the proposed road. So, fly levelling should be done to connect the BM with the starting point of the work in order to locate its RL and then calculate the RLs of different points along the alignment. The entries should be recorded as shown below.

Fly levelling from BM No. 1 at ____________ to the starting point of the proposed road from ____________ to ____________.

Station	BS	IS	FS	Rise (+)	Fall (–)	RL	Remark
BM	0.955					250.550	On BM No. 1
	1.250		2.150		1.195	249.355	
	0.785		1.760		0.510	248.845	
	1.535		2.055		1.270	247.575	
	1.260		0.835	0.700		248.275	
	0.675		0.955	0.305		248.580	
	1.275		1.505		0.830	247.750	
	1.655		2.050		0.775	246.975	
	0.450		2.160		0.505	246.470	
A			1.005		0.555	245.915	Starting point of road project
Total =	9.840		14.475	1.005	5.640		

Check: Σ BS – Σ FS = 9.840 – 14.475 = – 4.635
Σ Rise – Σ Fall = 1.005 – 5.640 = – 4.635
Last RL – 1st RL = 245.915 – 250.550 = – 4.635

4. Profile levelling (longitudinal levelling) To know the nature of the ground surface the profile levelling is done along the centre of the road at some regular intervals (say 20 m, 30 m, etc.) If required, staff readings may also be taken at points of importance where the slope of the ground suddenly changes.

Procedure: Suppose AB, BC, CD, and DE are the directions of the centre of the road. The levelling instrument is placed at suitable positions (say L_1, L_2, L_3, ...) and, after temporary adjustments, the staff readings are taken. The first staff reading of any set up is entered in the BS column, and the last in the FS column. The other readings are entered in the IS column. The fore bearing and back bearing of each line should be measured and entered in the level book.

Temporary bench-marks should be placed at some chainage intervals (say of 1,000 m) on the roots of trees or some permanent points. At the end of the day's work, a temporary bench-mark must be placed at a suitable point. Fly levelling (check levelling) should be done to connect this TBM to the starting point of the day's work. At the time of fly levelling, all the TBMs (which were placed previously) are also connected in order to detect the place of error. The next days work is started from the TBM placed on the previous day. The records pertaining to profile levelling are entered as follows (Fig. 5.27).

Check: Σ BS – Σ FS = 7.075 – 6.000 = + 1.075
Σ Rise – Σ Fall = 9.945 – 8.870 = + 1.075
Last RL – 1st RL = 246.990 – 245.915 = + 1.075

5. Cross-sectional Levelling While profile levelling is in progress, cross-sectional levelling should also be done. The cross-sections are taken perpendicular to the centre line of the alignment at some regular intervals (say 20 m, 40, etc). The purpose of cross-sectional levelling is to know the undulation of the ground surface transverse

Profile Levelling along the alignment of the proposed road from ______ to ______

Station	Chainage	Bearing FB	Bearing BB	Readings BS	Readings IS	Readings FS	Rise (+)	Fall (−)	RL	Remark
A	0	AB = 80°30′		1.525					245.915	Starting point of project C/S–1
	20				2.150			0.625	245.290	
	40				2.650			0.500	244.790	C/S–2
	60			0.950		0.850	1.800		246.590	CP
	80				2.055			1.105	245.485	C/S–3
	100				1.965		0.090		245.575	
B	115	BC = 120°30′	AB = 260°30′	1.305		1.255	0.710		246.285	CP
	120				1.850			0.545	245.740	C/S–4
	140				2.360			0.510	245.230	
	160			1.055		0.755	1.605		246.835	CP C/S–5
	180				1.860			0.805	246.030	
	200				2.950			1.090	244.940	
C	220	CD = 30°15′	BC = 300°30′	0.890		1.155	1.795		246.735	CP C/S–6
	240				1.755			0.865	245.870	
	260				2.680			0.925	244.945	
D	280	DE = 140°0′	CD = 210°15′	1.350		1.270	1.410		246.355	C/S–7
	300				2.105			0.755	245.600	
	320				2.655			0.550	245.050	C/S–8
	340				3.250			0.595	244.455	
E	360		DE = 320°0′		1.760		1.490		245.945	C/S–9
TBM						0.715	1.045		246.990	TBM kept on top of well
Total =				7.075		6.000	9.945	8.870		

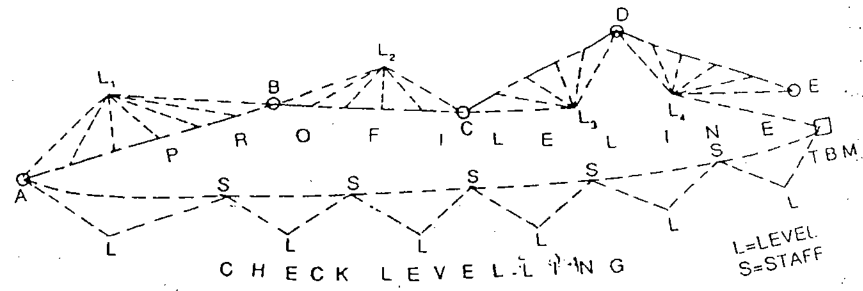


Fig. 5.27

to the centre of the road. The length of cross-section depends upon the nature of the work. In case of ordinary work, the length may be 20 or 40 m on each side of the centre line. The levels are taken at an interval of 5 m on each side. Additional readings may be taken if the nature of the ground surface suddenly changes.

The method of entering staff readings at a cross-section is shown below.

Cross-section 1 at chainage 0

Distances Left	Distances Centre	Distances Right	BS	IS	FS	Rise	Fall	RL	Remark
	0		0.760					245.915	Centre at
		5		1.875			1.115	244.800	chainage 0
		10		2.360			0.485	244.315	RL is taken
		15		0.985		1.375		245.690	from longi-
		20		0.375		0.610		246.300	tudinal
5				2.015			1.640	244.660	section
10				1.550		0.465		245.125	
15				0.790		0.760		245.885	
20					1.525		0.735	245.150	
Summation =			0.760		1.525	3.210	3.975		

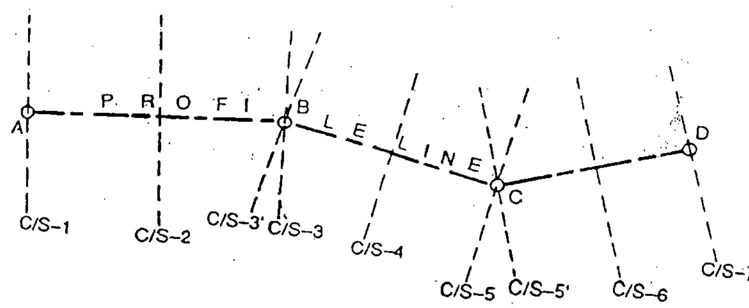


Fig. 5.28

Check: Σ BS – Σ FS = 0.760 – 1.525 = – 0.765
Σ Rise – Σ Fall = 3.210 – 3.975 = – 0.765
Last RL – 1st RL = 245.150 – 245.915 = – 0.765

Cross-section 2 at chainage 40

Distances			BS	IS	FS	Rise	Fall	RL	Remark
Left	Centre	Right							
	0		1.035					244.790	Centre at chainage 40. RL is taken from longitudinal section.
		5		2.620			1.585	243.205	
		10		3.155			0.535	242.670	
		15		1.935		1.220		243.890	
		20		0.760		1.175		245.065	
5				1.875			1.115	243.950	
10				2.620			0.745	243.205	
15				1.850		0.770		243.975	
20					0.975	0.875		244.850	
Total =			1.035		0.975	4.040	3.980	Last RL – 1st RL	
Check =			Σ BS – Σ FS = 1.035 – 0.975 = + 0.060			Σ Rise – Σ fall = 4.040 – 3.980 = + 0.060		= 244.850 – 244.790 = + 0.060	

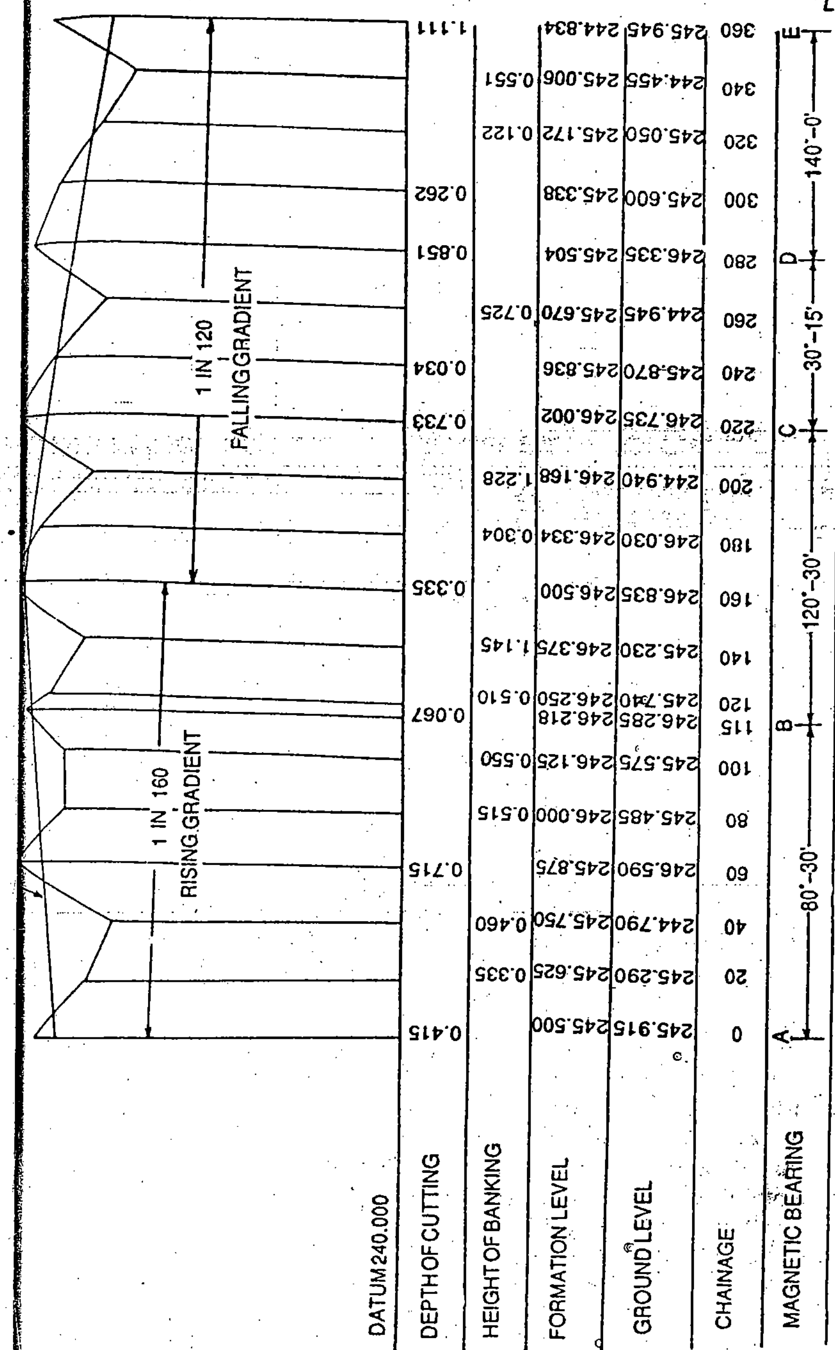
6. *Plotting of profile (longitudinal section)* Before plotting the longitudinal section, two scales are assumed. One is the horizontal scale which is normally of 1 : 1,000 or 1 : 2,000 and the other is the vertical scale which is of either 1 : 100 or 1 : 200. A horizontal line is drawn as the datum line. The chainages are marked along this line according to the horizontal scale. Then the ordinates (perpendicular lines) are drawn at each of the chainage points. The RL of the datum line is assumed in such a way that the ground surface can be shown above the datum. Now the vertical distances (RL of GL – RL of datum) are plotted along the ordinates according to the vertical scale. The plotted points are joined to obtain the outline of the ground surface, (as shown in Fig. 5.29). The formation line to be drawn in red ink. Other colour conventions are mentioned in subsection 9.

7. *Plotting of cross-section* The cross-sections are also plotted in the same way as the longitudinal section. But here the horizontal and vertical scales are slightly different. The horizontal and vertical scales are commonly adopted as 1 : 400 and 1 : 100 respectively (Fig. 5.30).

8. *Working profile* On the prepared longitudinal section, the formation level of the road is marked in such a way that cutting and banking may be equalised. Then the finished surface is also marked. This is known as the working profile.

The formation levels are calculated as follows.

Suppose a formation line of falling gradient 1 in 50 has been adopted.



HOR. SCALE–1:2000
VER SCALE–1:100

LONGITUDINAL SECTION
Fig. 5.29

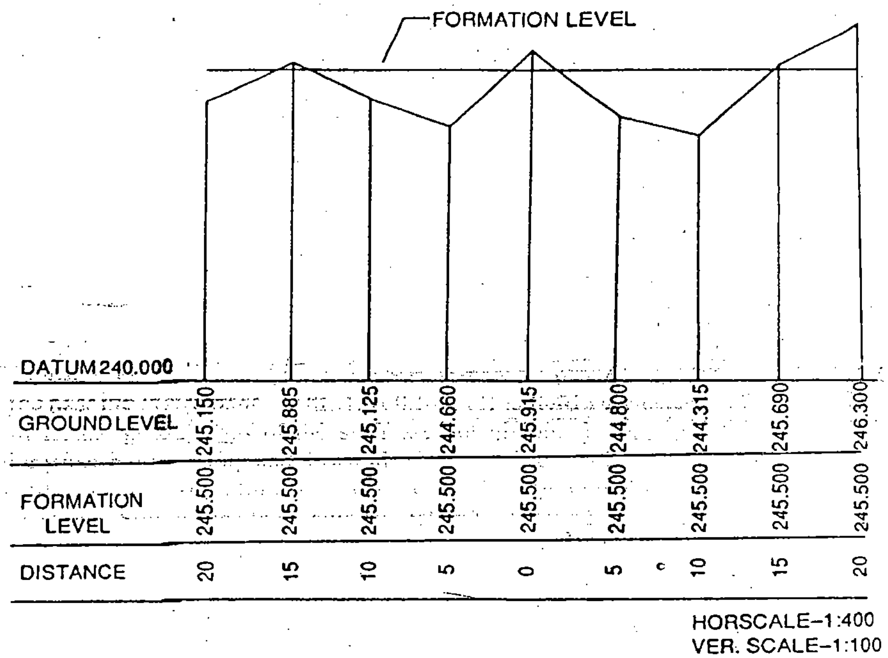


Fig. 5.30

Then Fall per 20 m = $\frac{20}{50}$ = 0.400 m

If Formation level at chainage 0 = 245.000 m

Then Formation level at chainage 20 = 245.000 – 0.400 = 244.600 m

Formation level at chainage 40 = 244.600 – 0.400 = 244.200 m

and so on.

9. Colour convention

(a) The datum, RL of the ground, and outlines of ground surfaces and chainages are to be taken in black ink.

(b) The ordinates are to be taken in blue lines.

(c) The formation line and levels are to be taken in red ink.

(d) The finished surface is to be shown in blue ink.

(e) The depth of cutting is to be written in red ink.

(f) The height of banking is entered in blue ink.

(g) The gradients (say 1 in 50) should be shown between the ordinates as red lines with black arrows at the ends.

(h) The bearings of lines should be shown between the stations as red lines with black arrows at the ends.

10. Curve design The alignment changes its direction in different places. The deflection angles are found out and necessary data for setting out horizontal circular curves are calculated and tabulated.

If vertical curves along the alignment are unavoidable, then necessary data for setting them out are also calculated and tabulated. (Curve setting is discussed in detail in Chapter 10.)

11. Computation of volume of earth work After completion of the necessary drawing work, the volume of earth work for the entire project is computed by the trapezoidal formula or the prismoidal formula (given in chapter 8).

Then the total cost of the project is worked out by considering different items of work as per government schedule of rates.

5.15 DIFFICULTIES FACED IN LEVELLING

1. When the staff is too near the instrument If the levelling staff is held very near the levelling instrument, the graduations of the staff are not visible. In such a case, a piece of white paper is moved up and down along the staff until the edge of the paper is bisected by the line of collimation. Then the reading is noted from the staff with the naked eye. Sometimes the reading is taken by looking through the object glass.

2. Levelling across a large pond or lake Suppose the levelling is to be done across a very wide pond or lake.

We know that the water surface of a still lake or pond is considered to be level. Therefore, all points on a water surface have the same RL. Two pegs A and B are fixed on opposite banks of the lake or pond. The tops of the pegs are just flush with the water surface. The level is set up at O_1 and the RL of A is determined by taking an FS on A. The RL of B is assumed to be equal to that of A. Now the level is shifted and set up at O_2. Then by taking a BS on peg B, levelling is continued (Fig. 5.31).

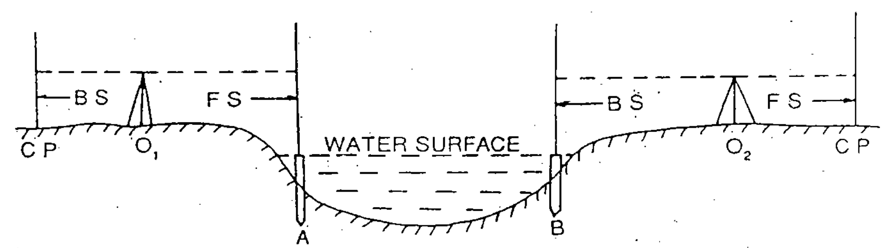


Fig. 5.31

3. Levelling across a river In case of flowing water, the surface cannot be considered level. The water levels on the opposite edges will be different. In such a case, the method of reciprocal levelling is adopted. Two pegs A and B are driven on the opposite banks of the river (not flush with the water surface). The RL of A is determined in the usual way. Then the true difference of level between A and B is found out by reciprocal levelling. Thus the RL of B is calculated, and levelling is continued.

4. Levelling across a solid wall When levelling is to be done across a brick wall, two pegs A and B are driven on either side of the wall, just touching it. The level is set up at O_1 and a staff reading is taken on A. Let this reading be AC. Then the height of the wall is measured by staff. Let the height be AE. The HI is found out by taking a BS on any BM or CP.

Then RL of A = HI − AC
RL of E = RL of A + AE = RL of F (same level)

The level is shifted and set up at O_2. The staff reading BD is noted and the height BF is measured.

Then RL of B = RL of F − BF
HI at O_2 = RL of B + BD

The levelling is then continued by working out the HI of the setting (Fig. 5.32).

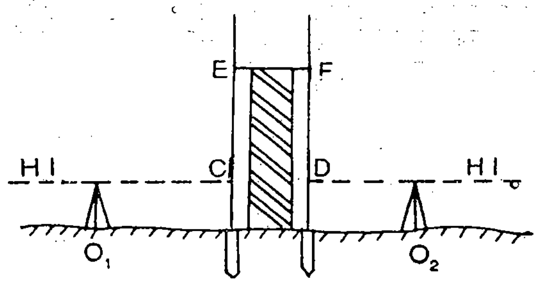


Fig. 5.32

5. When BM is above line of collimation This happens when the BM is at the bottom of a bridge girder or on the bottom surface of a culvert. It also happens when the RLs of points above the height of the line of collimation have to be found out.

Suppose the BM exists on the bottom surface of a culvert, and that it is required to find out the RL of A. The level is set up at O and the staff is held inverted on the BM. The staff reading is taken and noted with a negative sign. The remark "staff held inverted" should be entered in the appropriate column. Let the BS and FS readings be − 1.500 and 2.250 respectively (Fig. 5.33).

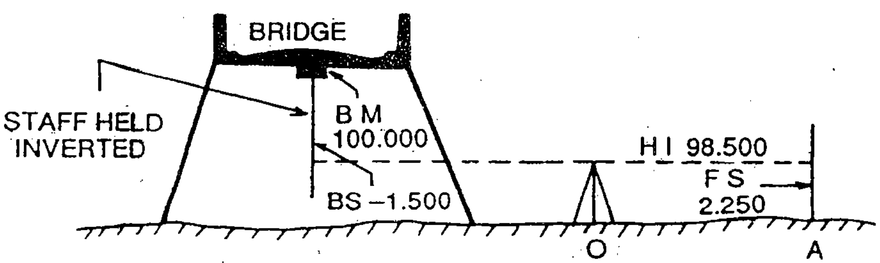


Fig. 5.33

Now, height of instrument = 100.000 − 1.500 = 98.500
RL of A = 98.500 − 2.250 = 95.250

6. Levelling along a steep slope While levelling along a steep slope in a hilly area, it is very difficult to have equal BS and FS distances. In such cases, the level should be set up along a zig-zag path so that the BS and FS distances may be kept equal. Let AB be the direction of levelling. I_1, I_2, ... are the positions of level and S_1, S_2, S_3, ... the positions of staff (Fig. 5.34).

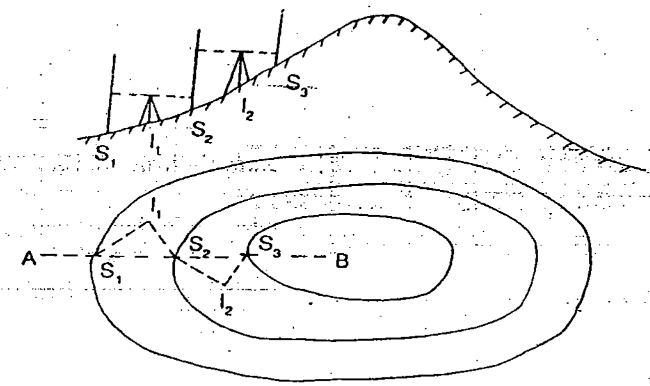


Fig. 5.34

Levelling is continued in this manner and the RLs of the points are calculated.

7. Levelling across a rising ground or depression While levelling across high ground, the level should not be placed on top of this high ground, but on one side so that the line of collimation just passes through the apex.

While levelling across a depression, the level should be set up on one side and not at the bottom of the depression (Figs 5.35(a) and (b)).

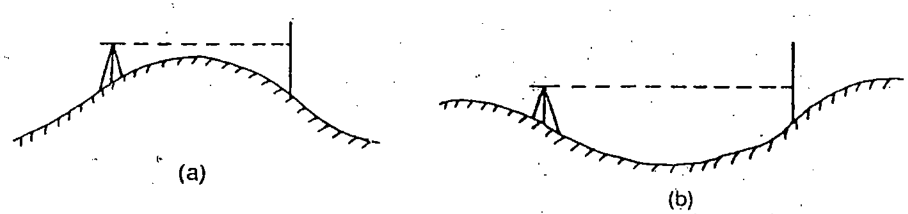


Fig. 5.35

5.16 SOURCES OF ERROR IN LEVELLING

The following are the different sources of error in levelling:

1. Instrumental errors

(a) The permanent adjustment of the instrument may not be perfect. That is, the line of collimation may not be parallel to the axis of the bubble tube.

(b) The internal arrangement of the focussing tube is not perfect.
(c) The graduation of the levelling staff may not be perfect.

2. Personal errors

(a) The instrument may not be levelled perfectly.
(b) The focussing of the eye-piece and object glass may not be perfect and the parallax may not be eliminated entirely.
(c) The position of the staff may be displaced at the change point at the time of taking FS and BS readings.
(d) The staff may appear inverted when viewed through the telescope. By mistake, the staff readings may be taken upwards instead of downwards.
(e) The reading of the stadia hair rather than the central collimation hair may be taken by mistake.
(f) A wrong entry may be made in the level book.
(g) The staff may not be properly and fully extended.

3. Errors due to natural causes

(a) When the distance of sight is long, the curvature of the earth may affect the staff reading.
(b) The effect of refraction may cause a wrong staff reading to be taken.
(c) The effect of high winds and a shining sun may result in a wrong staff reading.

5.17 PERMISSIBLE ERROR IN LEVELLING

The precision of levelling is ascertained according to the error of closure. The permissible limit of closing error depends upon the nature of work for which the levelling is to be done. Permissible closing error is expressed as

$$E = C\sqrt{D}$$

where, E = closing error in metres,
C = the constant, and
D = distance in kilometres.

The following are the permissible errors for different types of levelling:

1. Rough levelling —$E = \pm\, 0.100\sqrt{D}$
2. Ordinary levelling—$E = \pm\, 0.025\sqrt{D}$
3. Accurate levelling—$E = \pm\, 0.012\sqrt{D}$
4. Precise levelling —$E = \pm\, 0.006\sqrt{D}$

5.18 DETERMINATION OF STADIA CONSTANT

From the theory of the telescope it is known that

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

where, D = distance between vertical axis of telescope and staff,
f = focal length of object glass,
i = length of image,
s = difference of reading between the lower and upper stadia, and
d = distance between optical centre and vertical axis of telescope.

The quantity (f/i) is known as the multiplying constant and its value is usually 100. The quantity $(f + d)$ is called the additive constant and its value is normally zero. But sometimes its value lies between 20 and 30 cm.

The value of the constants are obtained by computation from field measurements.

Procedure:

1. A line OA, about 200 m long, is measured on level ground.
2. Pegs are fixed along this line at a known interval, say of 20 m.
3. The instrument is set up at O and stadia hair readings are taken at each of the pegs.
4. Thus the values of D and S are known for each of the pegs.
5. Putting these values of D and S in the equation,

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

we get a number of equations.

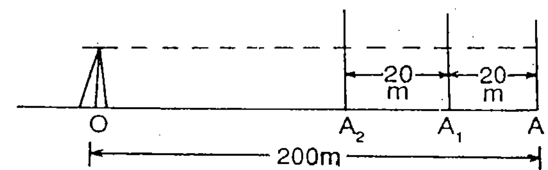


Fig. 5.36

6. The equations are solved in pairs to get several values of (f/i) and $(f + d)$. The mean of these values is taken as the stadia constant of the corresponding instrument.

Note: The theory of stadia will be discussed in Chapter 11.

Example The following field observations are made with a level

Distance in metres	Lower stadia	Upper stadia
200	1.645	3.645
180	1.050	2.850

Let us find the multiplying and additive constants.

Solution We know that

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

Let $\frac{f}{i} = x$ and $(f + d) = y$

$$D = xS + y \quad (1)$$

Substituting the observed values in (1), we get

$$200 = 2.000x + y \quad (2)$$

$$180 = 1.800x + y \quad (3)$$

Substracting Eq. (3) from Eq. (2)

$$20 = 0.200x$$

or $$X = \frac{20}{0.200} = 100$$

From Eq. (2),

$$Y = 200 - 2.000 \times 100 = 0$$

Therefore, Multiplying constant = $f/i = 100$

Additive constant $(f + d) = 0$

(a) In the first case, $S = 3.645 - 1.645 = 2.000$

(b) In the second case, $S = 2.850 - 1.050 = 1.800$

5.19 DETERMINING DISTANCE BY STADIA METHOD

Procedure

1. Suppose it is required to measure the distance AB, where chaining is not possible.
2. The level is set up at A and levelled properly.
3. The staff is held at B. After proper focussing (eliminating the parallax), the upper and lower stadia readings are taken.
4. The difference of the two staff readings is multiplied by the stadia constant (multiplying constant) to obtain the distance AB.
5. The stadia constant is normally 100. If it is not, it should be mentioned in the booklet supplied with the level. If there is any additive constant, that should also be mentioned in the booklet.

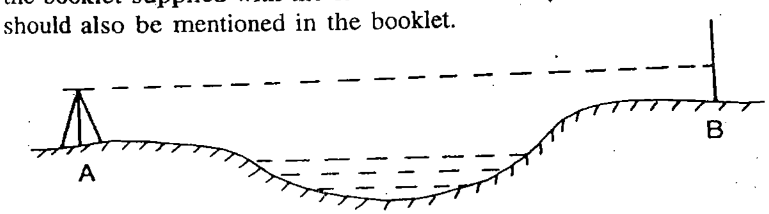


Fig. 5.37

Example Suppose the upper and lower stadia readings are 3.250 and 1.150 respectively.

Then, $AB = (3.250 - 1.150) \times 100 = 210$ m

5.20 PERMANENT ADJUSTMENT OF LEVEL

The establishment of a desired relationship between the fundamental lines of a levelling instrument is termed permanent adjustment. So, permanent adjustment indicates the rectification of instrumental errors.

The fundamental lines are as follows:

1. The line of collimation,
2. The axis of the bubble tube,
3. The vertical axis
4. The axis of the telescope.

The following relationships between the lines are desirable:

1. The line of collimation should be parallel to the axis of the bubble.
2. The line of collimation should coincide with the axis of the telescope.
3. The axis of the bubble should be perpendicular to the vertical axis. That is, the bubble should remain in the central position for all directions of the telescope.

Principle of reversal The principle of reversal states that if there is any error in a certain part of the instrument, then it will be doubled by reversing, i.e. by revolving the telescope through 180°. Thus the apparent error becomes twice the actual error on reversal.

Suppose, for example, that in a right-angled triangle ABC (Fig. 5.38), the angle ACB is not exactly 90° but less by $\theta°$. If this triangle is reverses as A_1B_1C, then the angle between the faces BC and B_1C becomes $2\theta°$. This is the underlying principle of reversal.

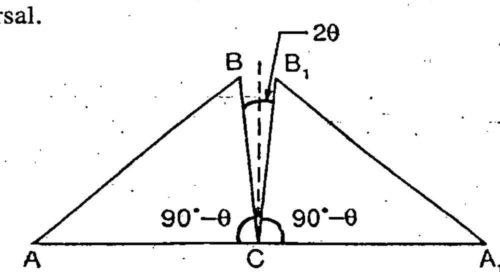


Fig. 5.38

This principle is followed in levelling instruments and theodolite. By the principle of reversal, the relationship between the fundamental lines can be determined and hence the necessary correction can be applied.

A. Permanent Adjustment of Dumpy Level

Two adjustments are required in the dumpy level.

1. The first adjustment, to make the axis of the bubble tube perpendicular to the vertical axis
2. The second adjustment, to make the line of collimation parallel to the axis of the bubble tube.

1. First adjustment The following procedure is adopted to make the line of collimation parallel to the axis of the bubble tube:

(a) The level is set up on fairly level and firm ground, with its legs well apart. It is firmly fixed to the ground.
(b) The telescope is placed parallel to any pair of foot screws and, by turning the foot screws either both inward or both outward, the bubble is brought to the centre.
(c) The telescope is then turned through 90°, so that it lies over the third foot screw. Then by turning the third foot screw the bubble is brought to the centre.
(d) The process is repeated several times until the bubble is in the central position in both the directions.
(e) Now the telescope is turned through 180° and the position of the bubble is noted.
If the bubble still remains in the central position, the desired relationship is perfect. If not, the amount of deviation of the bubble is noted.
(f) Suppose, the deviation is of $2n$ divisions. Now by turning the capstan headed nut (which is at one end of the tube), the bubble is brought half-way back (i.e. n division). The remaining half-deviation (i.e. n divisions) is adjusted by the foot screw or screws just below the telescope.
(g) The procedure of adjustment is continued till the bubble remains in the central position at any position of the telescope.

2. Second adjustment The second adjustment is done by two-peg method, which is described below.

(a) Two pegs A and B are driven at a known distance apart (say D) on level and firm ground. The level is set up at P, just mid-way between A and B. After bringing the bubble to the centre of its run (usual), the staff readings on A and B are taken. Suppose the readings are a and b.
Now the difference of level between A and B is calculated, this difference is the true difference, as the level is set up just mid-way between BS and FS (Fig. 5.39(a)).
Then the rise or fall is determined by comparing the staff readings.
(b) The level is shifted and set up at P_1 (very near A), say at a distance d from A. Then after proper levelling (following the usual method), staff readings at A and B are taken. Suppose the readings are a_1 and b_1.
Then the apparent difference of level is calculated (Fig. 5.39(b)).
(c) If the true difference and apparent difference are equal, the line of collimation is in adjustment. If not, the line of collimation is inclined.
(d) In the second set up, let e be the staff reading on B at the same level of the staff reading a_1.

Then $$e = a_1 \pm \text{true difference}$$

(Use the positive sign in the case of a fall and the negative sign when there is a rise.)

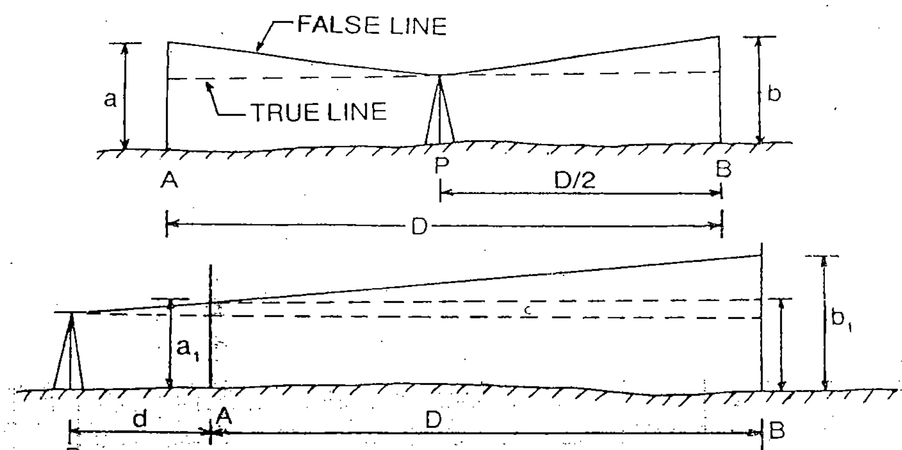


Fig. 5.39 (a) & (b)

(e) If b_1 is greater than e, the line of collimation is inclined upwards and if b_1 is less than e, it is inclined downwards.

$$\therefore \quad \text{Collimation error} = b_1 \sim e \quad \text{(in distance } D\text{)}$$

(f) By applying the principle of similar triangle

Correction to near peg, $C_1 = \frac{d}{D} \quad (b_1 \sim e)$

Correction to far peg, $C_2 = \frac{D+d}{D} \quad (b_1 \sim e)$

Correct staff reading on $A = a_1 \pm C_1$
Correct staff reading on $B = b_1 \pm C_2$

(Use the positive sign when the line of collimation is inclined downwards, and the negative sign when it is inclined upwards.)

(g) Then the cross-hair is brought to the calculated correct reading by raising or lowering the diaphragm by means of the diaphragm screw.

B. Field Instructions

1. If the correct reading is seen below the collimation hair on looking through the telescope, the cross-hair is to be lowered. This is done by loosening the upper screw and tightening the lower screw of the diaphragm.
2. If the correct reading is seen above the collimation hair, on looking through the telescope, the cross-hair should be raised. This is done by loosening the lower screw and tightening the upper screw of the diaphragm.

5.21 PROBLEMS ON PERMANENT ADJUSTMENT

Problem 1 The following observations were made during the testing of a dumpy level.

Instrument at	Staff reading at	
	A	B
A	1.725	2.245
B	2.145	3.045

Distance between A and B = 200 m
RL of A = 450.000 m

(a) Is the instrument is in adjustment?
(b) What should be the staff reading on A during the second set up of the instrument for the line of collimation to be exactly horizontal?
(c) To what reading should the line of collimation be adjusted when the instrument is at B?
(d) What should be the RL of B?

Solution When the instrument is at A:

Apparent difference of level between AB = (2.245 – 1.725)
= 0.520 m (fall from A to B)

When the instrument is at B:

Apparent difference of level = 3.045 – 2.145 = 0.900 m

(a) Since the two apparent differences of level are not equal, the line of collimation is not in adjustment.

$$\text{True difference of level} = \frac{0.520 + 0.900}{2} = 0.710 \text{ m}$$

(b) In the second set up,

True reading on B = 3.045 (As level is set up near B)
True reading on A = 3.045 – 0.710 = 2.335 m

(c) Collimation error = observed reading on A – true reading on A

= 2.145 – 2.335 = – 0.190 m

(Here, the error is negative. So the correction should be positive.)

(d) RL of B = RL of A – true difference

= 450.000 – 0.710 = 449.290 m

Problem 2 In a two-peg test of a dumpy level, the following readings were taken.

Instrument at	Staff reading on	
	A	B
Mid-way between A and B	1.585	1.225
A	1.425	1.150

Distance between the pegs A and B is 100 m.

(a) With the instrument at A, what should be the staff reading on B for the line of collimation to be exactly horizontal?
(b) Is the line of collimation inclined upwards or downwards?
(c) What is the amount of collimation error?

Solution When instrument is mid-way between A and B,

true difference of level = 1.585 – 1.225 = 0.360 m
(rise from A to B)

(a) When the instrument is at A,
Correct staff reading on A = 1.425 m (as the level was near A)
Correct staff reading on B should be = 1.425 – 0.360 = 1.065 m

(b) Since the observed staff reading on B is greater than the correct staff reading, the line of collimation is inclined upwards.

(c) Collimation error = observed reading – correct reading
= 1.150 – 1.065 = + 0.085 m

(Here, the error is positive. So the correction should be negative.)

Problem 3 A levelling instrument was set up exactly mid-way between two pegs A and B, 100 m apart. The staff readings on A and B were 1.875 and 1.790 respectively. The instrument was then set up at a distance of 10 m from A on the line AB. The respective staff readings were 1.630 and 1.560.

Calculate the correct staff reading on A and B when the line of collimation is exactly horizontal.

Solution When the level is mid-way between A and B,

True difference of level = 1.875 – 1.790 = 0.085 m (rise from A to B)

When the level is 10 m away from A,

Apparent difference of level = 1.630 – 1.560 = 0.070 m

Since the apparent difference is not equal to the true difference, the line of collimation is not in adjustment.

In second setting

Reading on B at same level of staff reading on A = 1.630 – 0.085 = 1.545 m

Since the observed reading (1.560) m on B is greater than the calculated value (i.e. 1.545), the line of collimation is inclined upwards.

Collimation error in 100 m = 1.560 – 1.545 = + 0.015 m

Since the error is positive, the correction to be applied is negative.

$$\text{Correction on A} = \frac{10}{100} \times (0.015) = 0.0015 \text{ m} \quad \text{(negative)}$$

$$\text{Correction on B} = \frac{110}{100} \times (0.015) = 0.0165 \quad \text{(negative)}$$

Correct reading on A = 1.6300 – 0.0015 = 1.6285 m

Correct reading on B = 1.5600 – 0.0165 = 1.5435 m

SHORT QUESTION WITH ANSWERS FOR VIVA

Q. 1 What is a datum surface?

Ans. A datum surface is an arbitrarily assumed level surface from which the vertical distances of various objects are measured.

Q. 2 What does the term GTS mean?

Ans. GTS means "Great Trigonometrical Survey".

Q. 3 What are bench-marks?

Ans. A reference point whose RL is fixed with respect to the datum surface is known as a bench-mark.

Q. 4 What is the datum adopted for GTS bench-marks?

Ans. The mean sea level at Karachi is adopted as the datum for GTS bench-marks. It is considered as "zero".

Q. 5 What are the types of BM that you know of?

Ans. Four types—(a) GTS BM, (b) permanent BM (c) the temporary BM, and (d) the arbitrary BM.

Q. 6 For any engineering work, how will you get the RL of the starting point?

Ans. The starting point is connected to the GTS or permanent BM by fly levelling. Then the RL of the starting point is calculated by the usual method.

Q. 7 What is the difference between a level surface and a horizontal surface?

Ans. A surface parallel to the mean spheroidal surface of the earth is known as a level surface. But a horizontal surface is tangential to the level surface at any point. The surface of a still lake is considered to be level. The surface perpendicular to the direction of gravity (indicated by the plumb line) is considered to be horizontal.

Q. 8 What is the difference between the line of collimation and axis of the telescope?

Ans. The line of collimation is the line joining the point of intersection of the cross-hairs to the optical centre of the object glass. The axis of the telescope is the line joining the optical centre of the object glass to that of the eye-piece.

Q. 9 What is the relation between the line of collimation and the axis of a telescope?

Ans. Both these lines should coincide.

Q. 10. In a particular set up of the level, suppose four readings are taken. How should they be entered in the level book?

Ans. The first reading should be entered in the BS column, the last reading in the FS column, and the other two readings in the IS column.

Q. 11 What is a change point?

Ans. Such a point indicates shifting of the instrument. At this point, a foresight reading is taken from one setting and a backsight reading from the next setting.

Q. 12 The staff readings on A and B are 1.735 and 0.965 respectively. Which point is higher?

Ans. Point B is higher.

Q. 13 What is the procedure of levelling by foot screws?

Ans. The telescope is first placed parallel to any pair of foot screws and the bubble is brought to the centre by turning the foot screws equally either inward or outward. Then the telescope is turned through 90° and the bubble is brought to the centre by turning the third foot screw. This process is repeated several times.

Q. 14 How is the level centred?

Ans. In a levelling operation, the level is never centred. It can be set up at any suitable position. The level is centred only when the magnetic bearing of any line is taken with the compass attached to the levelling instrument.

Q. 15 Suppose a level is given to you whose line of collimation is not in adjustment. What is the procedure that you would follow in order to work with this instrument?

Ans. The principle of equalising backsight and foresight distances should be followed. This means that the level should always be placed exactly mid-way between BS and FS.

Q. 16 How will you continue levelling across a river?

Ans. Reciprocal levelling should be undertaken across a river.

Q. 17 How will you continue levelling across a lake or pond?

Ans. We know that the water surface of a lake or pond is level. So, two pegs are fixed on opposite banks flush with the water surface. Then an FS reading is taken on one peg and the RL is calculated. After this, a BS reading is taken on the other peg. As the water surface is level, the RL of the second peg is assumed to be equal to that of the first peg, and the levelling operation is continued.

Q. 18 What are the arithmetical checks for the HI method and the rise-and-fall method?

Ans. The arithmetical check for the HI method is as follows:

$$\Sigma \text{ BS} - \Sigma \text{ FS} = \text{last RL} - \text{1st RL}$$

The arithmetical check for the rise-and-fall method is:

$$\Sigma \text{ BS} - \Sigma \text{ FS} = \Sigma \text{ rise} - \Sigma \text{ fall} = \text{last RL} - \text{1st RL}$$

Q. 19 What is fly levelling?

Ans. The levelling operation in which only BS and FS readings are taken and no intermediate sights are observed is known as fly levelling. Fly levelling is done for connecting the BM to the starting point of any project. In such levelling, no horizontal distances are required to be measured.

Q. 20 What is check levelling?

Ans. In case of longitudinal levelling, at the end of the day's work the finishing point is connected to the starting point of that days's work by fly levelling, to check the accuracy of the work. This operation is called check levelling.

Q. 21 What is a temporary bench-mark?

Ans. In case of longitudinal levelling, at the end of day's work, a bench-mark is kept at some suitable point. This bench-mark is called a temporary benchmark.

Q. 22 Why is datum assumed for plotting a levelling operation?

Ans. The RL of any point cannot be plotted to the full scale showing its full elevation. So, a datum (a reference line of assumed RL) is suitably assumed to show only the undulation of the ground surface.

Q. 23 What is the difference between temporary and permanent adjustment?

Ans. Temporary adjustment is done at every set up of the instrument before taking staff readings. Permanent adjustment is done in order to rectify any disturbed relationships between the fundamental lines.

Q. 24 What would you mean by positive RL and negative RL?

Ans. The vertical distance of a point above the datum surface is known as the positive RL, and the vertical distance of a point below the datum surface is said to be the negative RL.

Q. 25 How will you measure the distance between two points with only a level and staff?

Ans. The distance can be measured by the stadia method. The difference of the stadia hair readings is multiplied by 100 (stadia constant) to get the required distance.

EXERCISES

1. Define the following: datum surface, line of collimation, reduced level, bench-mark, change point, and parallax.
2. Describe a dumpy level along with a sketch.
3. What are the different types of levelling staves?
4. What is temporary adjustment? How is it done?
5. What are the different types of levelling operation?
6. Explain the principle of equalising backsight and foresight distances.
7. What are the different corrections applied to levelling?
8. What does the term 'sensitiveness' mean in the context of a bubble? How is the sensitiveness of a bubble is determined?
9. When is reciprocal levelling done? Describe the method along with a sketch.
10. What are the different methods of reduction of level? Which of them would you prefer?
11. What are the points to be remembered while entering records in a level book?
12. What are the sources of error in levelling?
13. What is permanent adjustment?
    Describe the two-peg method of adjustment along with a neat sketch.
14. In running fly levels from a bench-mark of RL 139.605, the following readings were obtained:

    Backsight—1.445, 2.595, 1.315, 2.825
    Foresight—0.475, 1.135, 0.495

    From the last position of the instrument, six pegs at 20-m intervals are to be set out on a uniform rising gradient of 1 in 50; the first peg is to have an RL of 143.000.

    Enter the readings on a level field book and work out the staff readings on the top of the pegs.
15. The following consecutive readings were taken with a level and 4-m levelling staff on a continuously sloping ground at common intervals of 30 m.

    0.905 (on A), 1.745, 2.345, 3.125, 3.725, 0.545, 1.390, 2.055, 2.955, 3.455, 0.595, 1.015, 1.850, 2.655, and 2.945 (on B).

    The RL of A was 395.500. Calculate the RLs of different points and find the gradient of the line AB.
16. The following readings are successively taken with a level:

    0.355, 0.485, 0.625, 1.755, 1.895, 2.350, 1.780, 0.345, 0.685, 1.230 and 2.150.

    The instrument was shifted after the fourth and seventh readings. Prepare a level book and calculate the RLs of different points. The RL of the first point is 255.500 m.
17. A page of a level book is shown in the following. Fill in the missing readings and calculate the RL of all points. Apply the usual checks.

Station	BS	IS	FS	Rise	Fall	RL	Remark
1	2.150					450.000	BM I
2	1.645		?	0.500			
3		2.345			?		
4	?		1.965	?			
5	2.050		1.825		0.400		
6	?		?	?		451.500	BM II
7	1.690		1.570	0.120			
8	2.865		2.100		?		
			?	?		451.250	BM III

18. Choose the correct alternative in questions (i) to (xx).
    (i) The datum adopted for India is the
    (a) MSL at Madras (b) MSL at Bombay (c) MSL at Karachi
    (ii) The BM established by the survey of India is known as the
    (a) Permanent BM (b) GTS BM (c) Arbitrary BM
    (iii) The surface of still water is considered to be
    (a) Level (b) Horizontal (c) Smooth
    (iv) The surface tangential to a level surface is said to be a
    (a) Vertical surface (b) Horizontal surface (c) Ground surface
    (v) The line of collimation and axis of the telescope should
    (a) Coincide (b) be parallel (c) be perpendicular
    (vi) The length of the sopwith telescopic levelling staff is
    (a) 3.5 m (b) 4 m (c) 5 m
    (vii) When there is relative movement between the cross-hairs and staff reading, it is known as
    (a) Parallax (b) Collimation error (c) Refraction error
    (viii) The staff reading taken on a point of known elevation is termed the
    (a) FS reading (b) BS reading (c) IS reading
    (ix) The internal focussing telescope is focussed by moving the
    (a) Convex lens (b) Double concave lens
    (c) Planoconvex lens
    (x) By arithmetical check, we can ensure the accuracy of
    (a) Field work (b) Calculation
    (c) Both field work and calculation
    (xi) The operation of levelling to determine the elevation between two points is known as
    (a) Simple levelling (b) Fly levelling (c) Differential levelling
    (xii) The BM fixed at the end of a days' work is called the
    (a) Permanent BM (b) Arbitrary BM (c) Temporary BM
    (xiii) The operation of levelling from the finishing point to the starting point at the end of a days' work is known as
    (a) Check levelling (b) Longitudinal levelling
    (c) Cross-sectional levelling
    (xiv) The operation of levelling across any river is termed
    (a) Profile levelling (b) Reciprocal levelling
    (c) Compound levelling
    (xv) The operation of levelling from any BM to the starting point of any project is known as
    (a) Longitudinal levelling (b) Fly levelling
    (c) Continuous levelling
    (xvi) To eliminate collimation error, the levelling instrument must be placed
    (a) Near the BS (b) Near FS
    (c) Exactly mid-way between the BS and the FS
    (xvii) The real image of the object is formed
    (a) In the plane of cross-hairs (b) At the centre of the eye-piece
    (c) At the centre of the telescope
    (xviii) The diaphragm is fitted
    (a) At the centre of the telescope (b) In front of the eye-piece
    (c) In front of the object glass
    (xix) The tangent to the longitudinal surface of the bubble tube is known as the
    (a) Axis of the bubble (b) Centre line of the bubble
    (c) Profile of the bubble

(xx) The sensitiveness of the bubble is directly related to
(a) The length of the bubble tube
(b) The radius of curvature of the bubble tube
(c) The cross-section of the bubble tube.

ANSWERS

18.	(i) c	(ii) b	(iii) a	(iv) b	(v) a
	(vi) b	(vii) a	(viii) b	(ix) b	(x) b
	(xi) c	(xii) c	(xiii) a	(xiv) b	(xv) b
	(xvi) c	(xvii) a	(xviii) b	(xix) a	(xx) b

6 Contouring

6.1 DEFINITIONS

1. Contour line The line of intersection of a level surface with the ground surface is known as the contour line or simply the contour. It can also be defined as a line passing through points of equal reduced levels.

For example, a contour of 100 m indicates that all the points on this line have an RL of 100 m. Similarly, in a contour of 99 m, all points have an RL of 99 m, and so on (Fig. 6.1).

A map showing only the contour lines of an area is called a contour map.

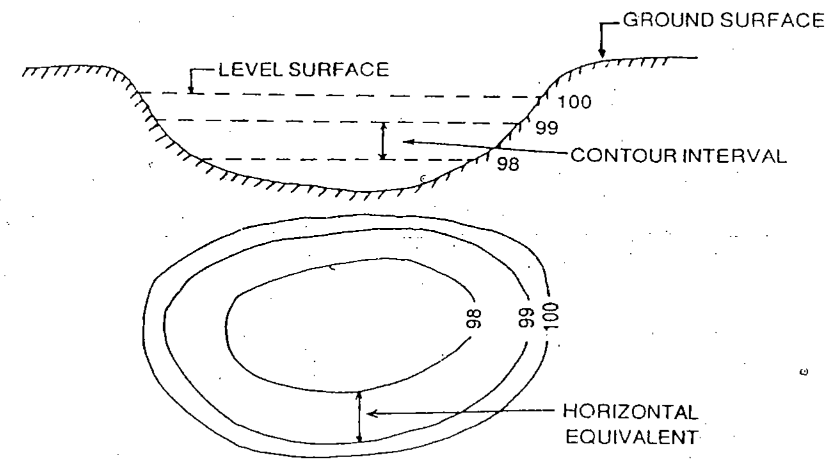


Fig. 6.1

2. Contour interval The vertical distance between any two consecutive contours is known as a contour interval. Suppose a map includes contour lines of 100 m, 98 m, 96 m, and so on. The contour interval here is 2 m. This interval depends upon: (i) the nature of the ground (i.e. whether flat or steep), (ii) the scale of the map, and (iii) the purpose of the survey.

Contour intervals for flat country are generally small, e.g. 0.25 m, 0.50 m, 0.75 m, etc. The contour interval for a steep slope in a hilly area is generally greater, e.g. 5 m, 10 m, 15 m, etc.

Again, for a small-scale map, the interval may be of 1 m, 2 m, 3 m, etc. and for large scale map, it may be of 0.25 m, 0.50 m, 0.75 m, etc.

It should be remembered that the contour interval for a particular map is constant.

3. Horizontal equivalent The horizontal distance between any two consecutive contours is known as horizontal equivalent. It is not constant. It varies according to the steepness of the ground.

For steep slopes, the contour lines run close together, and for flatter slopes they are widely spaced.

6.2 OBJECT OF PREPARING CONTOUR MAP

The general map of a country includes the locations of roads, railways, rivers, villages, towns, and so on. But the nature of the ground surface cannot be realised, from such a map. However, for all engineering projects involving roads, railways, and so on, a knowledge of the nature of ground surface is required for locating suitable alignments and estimating the volume of earth work. Therefore, the contour map is essential for all engineering projects. This is why contour maps are prepared.

6.3 USES OF CONTOUR MAP

The following are the specific uses of the contour map:

1. The nature of the ground surface of a country can be understood by studying a contour map. Hence, the possible route of communication between different places can be demarcated.
2. A suitable site or an economical alignment can be selected for any engineering project.
3. The capacity of a reservoir or the area of a catchment can be approximately computed.
4. The intervisibility or otherwise of different points can be established.
5. A suitable route for a given gradient can be marked on the map.
6. A section of the ground surface can be drawn in any direction from the contour map.
7. Quantities of earth work can be approximately computed.

6.4 CHARACTERISTICS OF CONTOURS

1. In Fig. 6.2, the contour lines are closer near the top of a hill or high ground and wide apart near the foot. This indicates a very steep slope towards the peak and a flatter slope towards the foot.

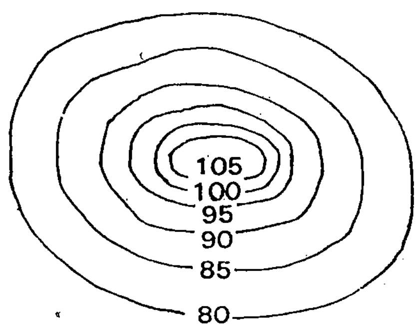


Fig. 6.2 Hill

2. In Fig. 6.3, the contour lines are closer near the bank of a pond or depression and wide apart towards the centre. This indicates a steep slope near the bank and a flatter slope at the centre.
3. Uniformly spaced, contour lines indicate a uniform slope (Fig. 6.4).
4. Contour lines always form a closed circuit. But these lines may be within or outside the limits of the map (Fig. 6.5).
5. Contour lines cannot cross one another, except in the case of an overhanging cliff. But the overlapping portion must be shown by a dotted line (Fig. 6.6).
6. When the higher values are inside the loop, it indicates a ridge line. Contour lines cross ridge lines at right angles (Fig. 6.7).
7. When the lower values are inside the loop, it indicates a valley line. Contour lines cross the valley line at right angles (Fig. 6.8).
8. A series of closed contours always indicates a depression or summit. The lower values being inside the loop indicates a depression and the higher values being inside the loop indicates a summit. (Fig. 6.9).
9. Depressions between summits are called saddles (Fig. 6.10 (b)).
10. Contour lines meeting at a point indicate a vertical cliff (Fig. 6.10 (a)).

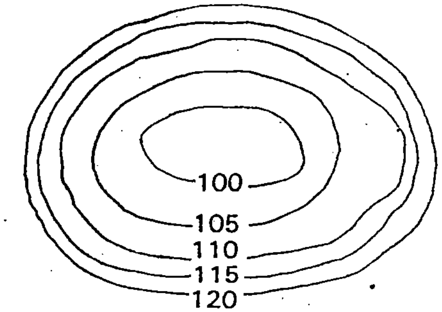


Fig. 6.3 Depression

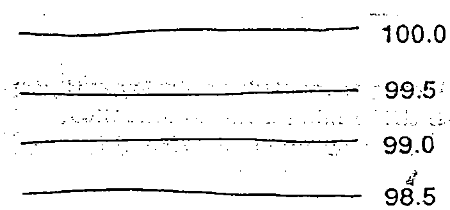


Fig. 6.4 Uniform Slope

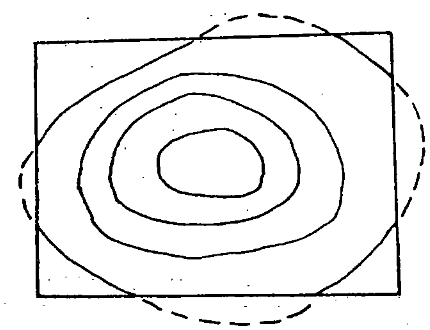


Fig. 6.5 Contour Closed within Map

6.5 METHODS OF CONTOURING

There are two methods of contouring—direct, and indirect.

A. Direct Method

There may be two cases, as outlined below.

Case I When the area is oblong and cannot be controlled from a single station:

In this method, the various points on any contour are located on the ground by

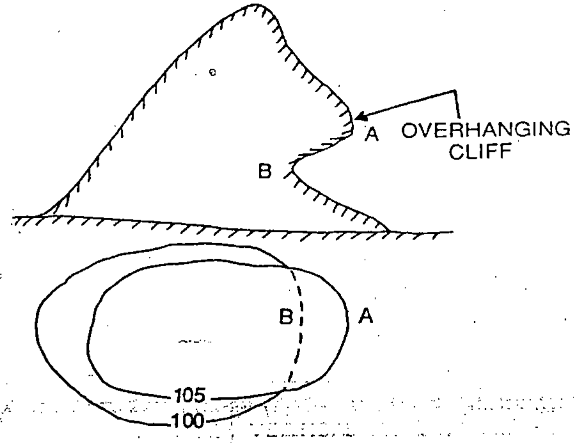


Fig. 6.6 Overhanging Cliff

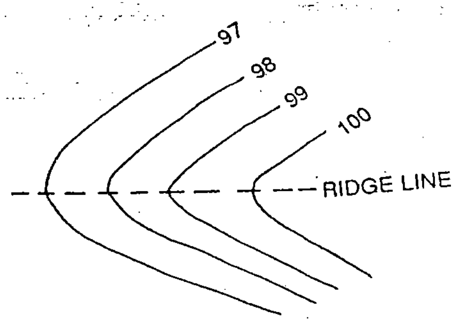


Fig. 6.7 Ridge Line

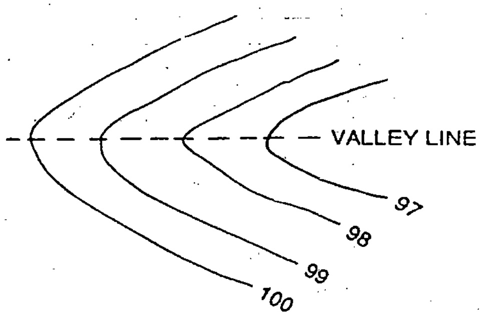
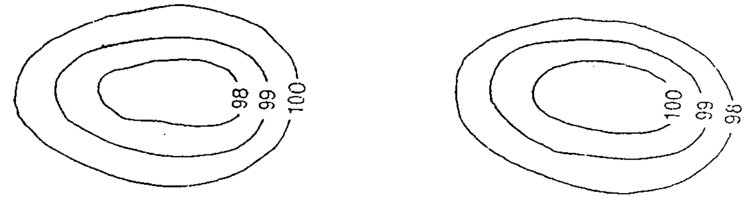


Fig. 6.8 Valley Line

taking levels. Then these points are marked by pegs. After this, the points are plotted on the map, to any suitable scale, by plane table. This method is very slow and tedious. But it gives accurate contour lines.

***Procedure*:**

1. Suppose a contour map is to be prepared for an oblong area. A temporary



(a) DEPRESSION (b) SUMMIT

Fig. 6.9

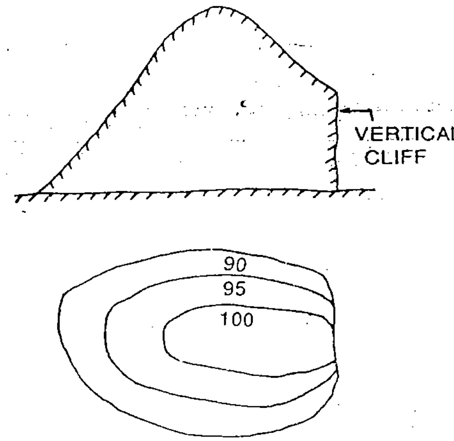


Fig. 6.10 (a) Vertical Cliff

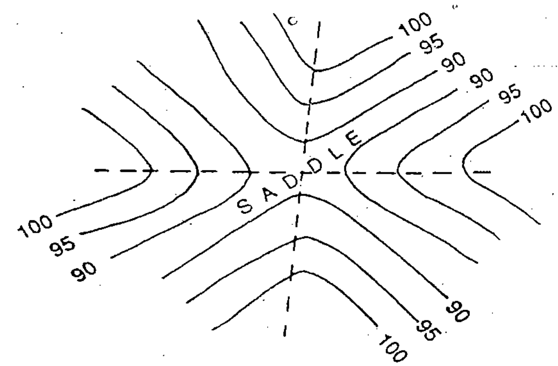


Fig. 6.10 (b) Saddle

bench-mark is set up near the site by taking fly level readings from a permanent bench-mark.
2. The level is then set up at a suitable position L from where maximum area can be covered.
3. The plane table is set up at a suitable station P from where the above area can be plotted.

4. A backsight reading is taken on the TBM. Suppose the RL of the TBM is 249.500 m and that the BS reading is 2.250 m. Then the RL of HI is 251.750 m. If a contour of 250.000 m is required, the staff reading should be 1.750 m. If a contour of 249.000 m is required, the staff reading should be 2.750 m, and so on.
5. The staffman holds the staff at different points of the area by moving up and down, or left and right, until the staff reading is exactly 1.750. Then the points are marked by pegs. Suppose, these points are A, B, C, D ...
6. A suitable point p is selected on the sheet to represent the station P. Then, with the alidade touching p, rays are drawn to A, B, C and D. The distances PA, PB, PC and PD are measured and plotted to a suitable scale. In this manner, the points a, b, c and d of the contour line of RL 250.000 m are obtained. These points are joined to obtain the contour of 250.000 m (Fig. 6.11).

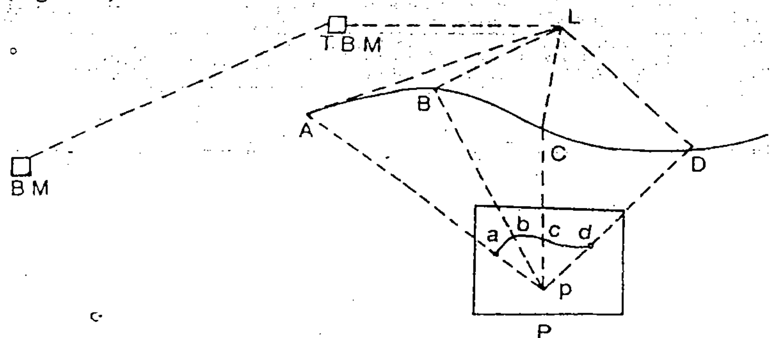


Fig. 6.11

7. Similarly, the points of the other contours are located.
8. When required, the levelling instrument and the plane table are shifted and set up in a new position in order to continue the operation along the oblong area.

Case II When the area is small and can be controlled from a single station: In this case, the method of radial lines is adopted to obtain contour map. This is also very slow and tedious, but gives the actual contour lines.

Direct Method

1. The plane table is set up at a suitable station P from where the whole area can be commanded.
2. A point p is suitably selected on the sheet to represent the station P. Radial lines are then drawn in different directions.
3. A temporary bench-mark is established near the site. The level is set up at a suitable position L and a BS reading is taken on the TBM. Let the HI in this setting be 153.250 m. So, to find the contour of RL 152.000 m a staff reading of 1.250 m is required at a particular point, so that the RL of contour of that point comes to 152.000 m

$$\text{RL} = \text{HI} - \text{Staff reading}$$
$$= 153.250 - 1.250 = 152.000 \text{ m}$$

4. The staffman holds the staff along the rays drawn from the plane table station in such a way that the staff reading on that point is exactly 1.250. In this manner, points A, B, C, D and E are located on the ground, where the staff readings are exactly 1.250.
5. The distances PA, PB, PC, PD and PE are measured and plotted to any suitable scale. Thus the points a, b, c, d and e are obtained which are joined in order to obtain a contour of 152.000.
6. The other contours may be located in similar fashion (Fig. 6.12).

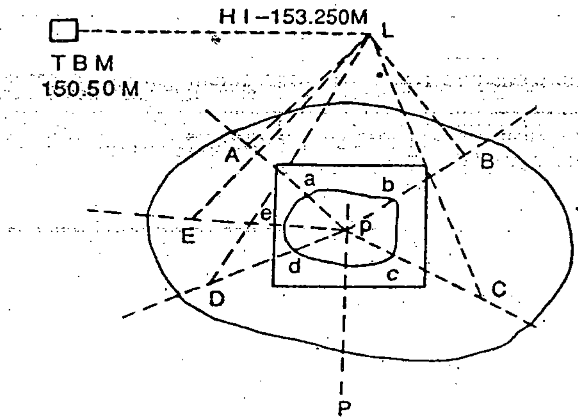


Fig. 6.12

Indirect Method In this method, the RLs of different points (spot levels) are taken at regular intervals along a series of lines set up on the ground. The positions of these points are plotted on a sheet to any suitable scale. The spot levels are noted at the respective points. Then the points of contour lines are found out by interpolation, after which they are joined to get the required contour lines. Although very quick, this method gives only the approximate positions of the contour lines. This method can be adopted in two ways, (i) cross-sections, and (ii) squares.

(a) Using cross-sections In this method, a base line, centre line or profile line is considered. Cross-sections are taken perpendicular to this line at regular intervals (say 50 m, 100 m etc.). After this, points are marked along the cross-sections at a regular intervals (say, 5m, 10m, etc). A temporary bench-mark is set up near the site. Staff readings are taken along the base line and the cross-sections. The readings are entered in the level book; the base line and the cross-sections should also be mentioned. The RL of each of the points calculated. Then the base line and cross-sections are plotted to a suitable scale. Subsequently the RLs of the respective points are noted on the map, after which the required contour line is drawn by interpolation (which is described in Sec. 6.6).

This method is suitable for route survey, when cross-sections are taken transverse to the longitudinal section (Fig. 6.13).

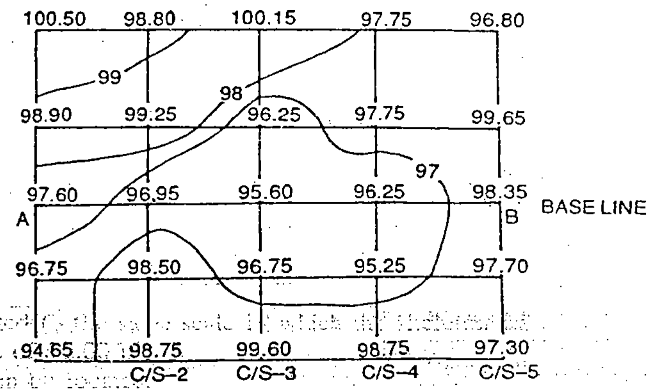


Fig. 6.13

(b) Using squares In this method (Fig. 6.14), the area is divided into a number of squares. The size of these squares depends upon the nature and extent of the ground. Generally, they have a sides varying from 5 to 20 m. The corners of the squares are numbered serially, as 1, 2, 3 ... A temporary bench-mark is set up near the site, and the level is set up at a suitable position. The staff readings on the corners of the squares are taken and noted in the level book maintaining the sequence of the serial numbers of the corners. The RLs of all the corners are calculated. The skeletons of the squares are plotted to a suitable scale. The respective RLs are noted on the corners, after which the contour lines are drawn by interpolation.

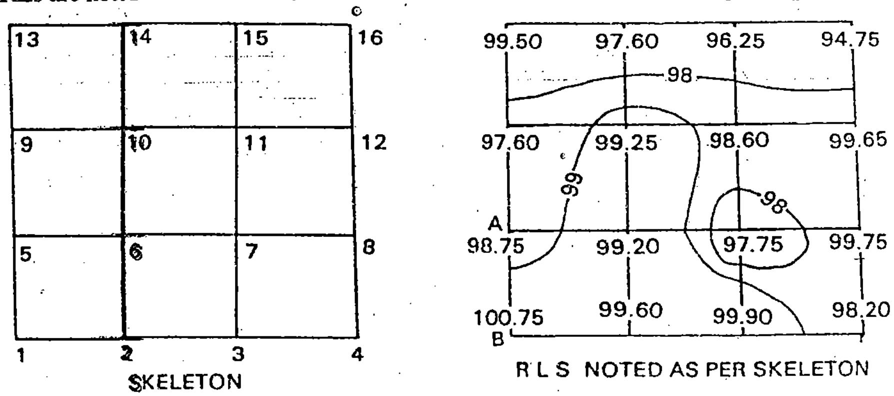


Fig. 6.14

6.6 METHOD OF INTERPOLATION OF CONTOURS

The process of locating the contours proportionately between the plotted points is termed interpolation. Interpolation may be done by:

1. Arithmetical calculation
2. The graphical method

1. By arithmetical calculation Let A and B be two corners of the squares (Fig. 6.14). The RL of A is 98.75 m, and that of B 100.75 m. The horizontal distance between A and B is 10 m.

Horizontal distance between A and B = 10 m

Vertical difference between A and B = 100.75 – 98.75 = 2 m

Let a contour of 99.00 m be required. Then,

Difference of level between A and 99.00 m contour = 99.00 – 98.75 = 0.25 m

$\therefore$ Distance of 99.00 m contour line from A = $\frac{10}{2} \times 0.25 = 1.25$ m

This calculated distance is plotted to the same scale in which the skeleton was plotted, to obtain a point of RL of 99.00 m.

Similarly, the other points can be located.

2. By graphical method On a sheet of tracing paper, a line AB is drawn and divided into equal parts (Fig. 6.15). AB is bisected at C, and a perpendicular is drawn at this point. A point O is selected on this perpendicular. Then radial lines are drawn from O to the divisions on AB. After this lines 1–1, 2–2, 3–3 ... are drawn parallel to AB. These lines serve as guide lines. The boundary line and every fifth line is marked with a thick or red line.

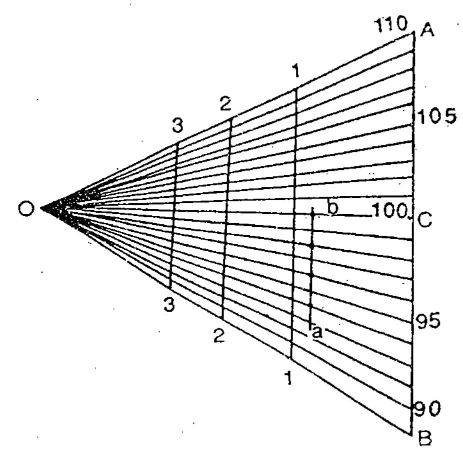


Fig. 6.15

Suppose we have to interpolate a 2 m contour interval between two points a and b of RLs 92.5 and 100.75 m.

Let us consider the lowest radial line OB to represent an RL of 90.00. So, every fifth line (which is bold or red) will represent 95,100, 105, etc. The tracing paper is moved over the plan until a lies at 92.5 and b at 100.25. Line ab should be parallel to AB. Now the points 94, 96, 98, 100 are pricked through to obtain the positions of the required contour.

6.7 CONTOUR GRADIENT

During preliminary survey for roads in a hilly area, the required points are first established along the gradient. The line joining these points is known as the contour gradient or grade contour.

Initially, the points are established approximately by an abney level, and then accurately fixed by a levelling instrument.

1. Location of contour gradient Suppose it is required to locate the centre line of a road in a hilly area with a ruling gradient of 1 in 20. Let the starting point A be on a 94.00 m contour line (Fig. 6.16). Since the contour interval is 2 m and gradient 1 in 20, the horizontal distance between A and the point on the next contour (96.00 m) is 2 × 20 = 40 m. With the centre at A and radius equal to 40 m (taken on the same scale), an arc is drawn cutting the contour line of 96.00 at point B.

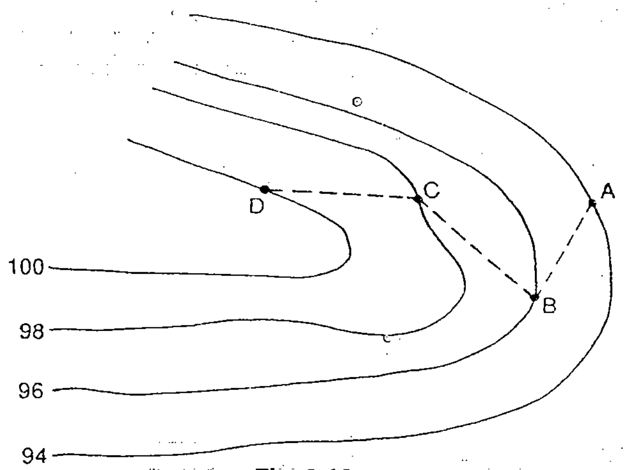


Fig. 6.16

Taking B as the centre and with the same radius, another arc is drawn to get the next point C. The other points are located in a similar manner.

6.8 FIELD LOCATION OF GRADE CONTOUR

1. By Abney level The Abney level (Fig. 6.17) is nothing but an improved type of clinometer. It consists of a telescope and spirit bubble. A mirror is provided over the bubble at an angle of 45° to help observe the image of the bubble. The bubble tube is attached to the vernier arm which can be rotated by a worm-and-wheel arrangement.

To fix the contour gradient, the index of the vernier is set to the angle corresponding to the given gradient. The Abney level is held over the starting station A against a pole at a suitable height C. A mark D is made on another pole at the same height. This pole is held over the next point of gradient. It is made

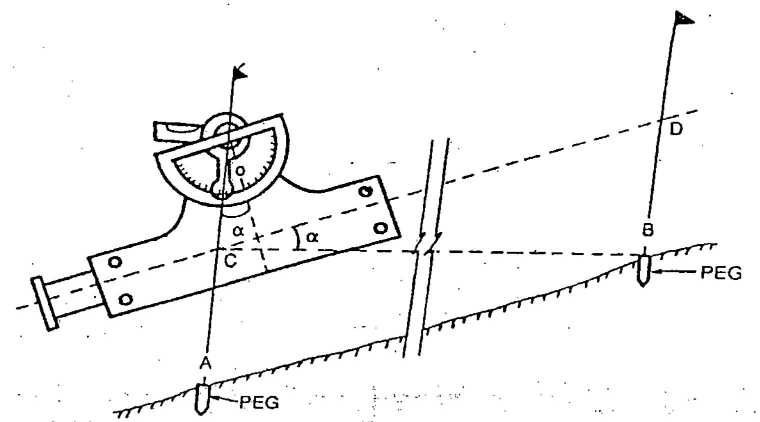


Fig. 6.17

to move up or down along the hill slope until the mark D is bisected at point B; at the same time the bubble should be at the centre of its run. Now the line joining the station point A to the point B is practically parallel to the line of sight and therefore on the given gradient. The points A and B are marked on the ground with pegs.

The Abney level is then shifted and held on the point D. The next point of the gradient is fixed according to the previous procedure. The other points are fixed on the gradient in similar fashion.

2. By levelling instrument In this method, the RL of the starting point is first determined with reference to the permanent bench-mark. Then the level is set up at a suitable position and a BS reading is taken on the starting point. Thus the HI is calculated for this setting. Then, by knowing the gradient and the peg interval, the RLs of the successive pegs are calculated. After this, the required staff readings on the pegs are determined. The locations corresponding to the calculated staff readings are identified and the points marked on the ground with pegs.

Example Suppose the RL of the starting point 525.50 m, the falling gradient 1 in 20 and peg interval 30 m.

Let the BS reading on the starting point be 1.525 m.

Then, HI = 525.500 + 1.525 = 527.025 m

$$\text{RL of next point} = 525.500 - \frac{30}{20} = 525.500 - 1.50 = 524.000 \text{ m}$$

$$\text{Staff reading on next peg} = 527.025 - 524.00 = 3.025 \text{ m}$$

Now, the staff is held at a distance of 30 m in such a way that the staff reading becomes exactly 3.025 m. The point is then marked with a peg. The process is repeated until all the points are located and marked with pegs.

SHORT QUESTIONS WITH ANSWERS FOR VIVA

Q. 1 What is a contour line?

Ans. A line joining points of equal elevation is known as a contour line.

Q. 2 Define the terms 'contour interval' and 'horizontal equivalent'?

Ans. The vertical distance between two consecutive contours is called a contour interval. The horizontal distance between two consecutive contours is known as horizontal equivalent.

Q. 3 Why is the horizontal equivalent not constant?

Ans. The horizontal distance between points varies according to the variation of slope. As the slope of the ground between two consecutive contours is not constant in all directions, the horizontal equivalent is not constant.

Q. 4 In some places consecutive contours run close together and in some places they are wide apart. What does this mean?

Ans. Contours running close together indicate a steep slope. When they run wide apart, it indicates a flatter slope.

Q. 5 How will you distinguish between a depression and a summit by studying the nature of the contour?

Ans. In case there is a depression, the contours run close together near the bank (outside) and wide apart at the centre. In the case of a summit, the contours run close together near the peak (centre) and wide apart at the base (outside).

Q. 6 In a map, it is found that two consecutive contours cross each other. What would you comment?

Ans. In general, contour lines cannot cross each other, except in the case of an overhanging cliff. Therefore, the area represented in the map includes an overhanging cliff. But the contour line should be dotted line at the point of crossing to indicate that one location is below the other.

Q. 7 How will you distinguish between a valley line and a ridge line?

Ans. When the lower values are inside the loop, it indicates a valley line. When the higher values are inside the loop, it indicates a ridge line.

Q. 8 What is a contour gradient?

Ans. In a hilly area, the centre line of a road in a given gradient is marked by some points. The line joining such points is known as contour gradient.

Q. 9 What is the object of preparing a contour map?

Ans. From a contour map, the nature of the ground surface of an area can be known. So, for identifying a suitable site for a dam or reservoir and for marking the tentative alignment of engineering projects involving roads, railways, etc., a contour map is essential.

EXERCISES

1. Define the terms 'contour line', 'contour interval' and 'horizontal equivalent'.
2. What are the characteristics of contour lines?
3. Show with neat sketches the characteristic features of contour lines of the following: (i) a pond, (ii) a hill, (iii) a ridge, (iv) A valley, and (v) a vertical cliff.
4. State the uses of a contour map.
5. What are the different methods of contouring? Describe any method along with sketch.
6. What is a grade contour? Describe, along with a sketch, how you would locate one in the field.

7. Describe the methods of interpolation of contours.
8. Choose the correct alternative in question (i) to (xv)
   (i) The line joining points of equal elevation is known as a
   (a) Horizontal line (b) Contour line (c) Level line
   (ii) The vertical distance between two adjacent contour lines is called a
   (a) Contour gradient (b) Vertical equivalent (c) Contour interval
   (iii) The line formed along the intersection of the ground surface and a level surface is known as a
   (a) Contour line (b) Watershed line (c) Level line
   (iv) A contour line intersects a ridge line or valley line
   (a) Obliquely (b) Perpendicularly (c) Vertically
   (v) The contour interval for a particular map is
   (a) Kept constant (b) Made variable (c) Made irregular
   (vi) When contour lines touch one another at a particular zone, it indicates a
   (a) Level surface (b) Vertical cliff (c) Horizontal surface
   (vii) When lower values are inside the loop, it indicates
   (a) High ground (b) Level ground (c) A depression
   (viii) When the higher values are inside the loop, it indicates a
   (a) Hill (b) Pond (c) Sloping ground
   (ix) The contour interval is inversely proportional to the
   (a) Steepness of the area (b) Extent of the area
   (c) Scale of the map
   (x) When a contour interval is fixed between 0.25 and 0.50 m, it indicates
   (a) A steep slope (b) A flattish slope (c) Almost level ground
   (xi) The alignments of highways are generally taken along
   (a) The ridge line (b) The valley line
   (c) Across the contour line
   (xii) When contours of different elevation cross each other, it indicates a/an
   (a) Vertical cliff (b) Saddle (c) Overhanging cliff
   (xiii) The horizontal distance between two consecutive contours is termed a
   (a) Contour interval (b) Horizontal equivalent
   (c) Horizontal interval
   (xiv) When consecutive contour lines run close together, it indicates a
   (a) Steep slope (b) Flatter slope (c) Vertical surface

ANSWERS

8. (i) b (ii) c (iii) a (iv) b (v) a
(vi) b (vii) c (viii) a (ix) a (x) b
(xi) a (xii) c (xiii) b (xiv) a

7

Computation of Area

7.1 INTRODUCTION

The term 'area' in the context of surveying refers to the area of a tract of land projected upon the horizontal plane, and not to the actual area of the land surface.

Area may be expressed in the following units:

1. Square-metres
2. Hectares (1 hectare = 10,000 m^2)
3. Square-feet
4. Acres (1 acre = 4840 sq. yd. = 43.560 sq. ft.)

The following is a hierarchical representation of the various methods of computation of area.

- Area
  - A. Graphical Method
    - From field notes
    - From plotted plan
      - Entire area
      - Boundary area
        - Mid-ordinate rule
        - Average ordinate rule
        - Trapezoidal rule
        - Simpson's rule
  - B. Instrumental Method

7.2 COMPUTATION OF AREA FROM FIELD NOTES

This is done in two steps.

Step 1 In cross-staff survey, the area of field can be directly calculated from field notes. During survey work the whole area is divided into some geometrical figures, such as triangles, rectangles, squares, and trapeziums, and then the area is calculated as follows:

1. Area of triangle = $\sqrt{s(s-a)(s-b)(s-c)}$

   where a, b and c are the sides,

   and $s = \dfrac{a+b+c}{2}$

   or Area of triangle = $1/2 \times b \times h$

   where b = base

   and h = altitude
2. Area of rectangle = $a \times b$

   where a and b are the sides
3. Area of square = a^2

   where a is the side of the square
4. Area of trapezium = $1/2\ (a + b) \times d$

   where a and b are the parallel sides, and d is the perpendicular distance between them.

Step 2 Consider Fig. 7.1. The area along the boundaries is calculated as follows

o_1, o_2 = ordinates

x_1, x_2 = chainages

$$\text{Area of shaded portion} = \frac{o_1 + o_2}{2} \times (x_2 - x_1)$$

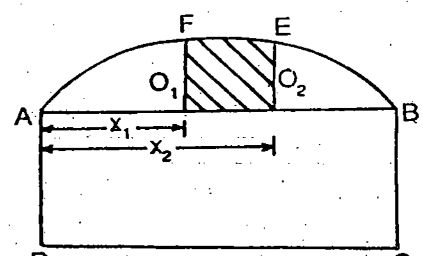


Fig. 7.1

Similarly, the areas between all pairs of ordinates are calculated and added to obtain the total boundary area.

Hence Total area of the field = area of geometrical figure + boundary areas (step 1 + step 2)

= area of ABCD + area of ABEFA

7.3 PROBLEMS ON COMPUTING AREA FROM FIELD NOTES

Problem 1 A page of the field book of a cross-staff survey is given in Fig. 7.2. Plot the required figure and calculate the relevant area.

Solution The figure is plotted as follows (Fig. 7.3).

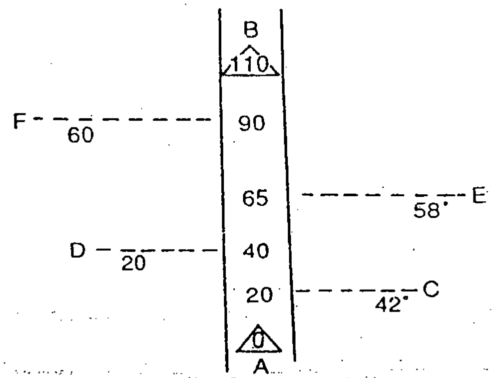


Fig. 7.2

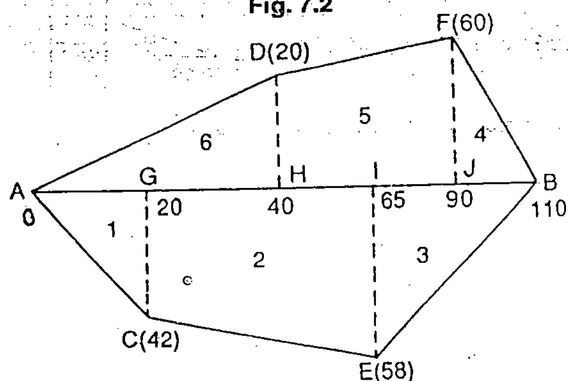


Fig. 7.3

The result is given in the following table:

Sl No.	Figure	Chainage (m)	Base (m)	Offset (m)	Mean offset(m)	Area (m^2) +ve	Area (m^2) −ve	Remark
1	2	3	4	5	6	7	8	9
1	ΔACG	0 and 20	20	0 and 42	21	420	—	Area = col 4 × col 6
2	Trap. GCEI	20 and 65	45	42 and 58	50	2,250	—	
3	ΔIEB	65 and 110	45	58 and 0	29	1,305	—	
4	ΔBFJ	90 and 110	20	0 and 60	30	600	—	
5	Trap. FJHD	40 and 90	50	60 and 20	40	2,000	—	
6	ΔDHA	0 and 40	40	20 and 0	10	400	—	
						6,975		

Area of field = 6975 sq. m

Problem 2 Figure 7.4 shows the page of a field book of a cross-staff survey. Plot the required figure and calculate the area of the trapezium ABCDEFA.

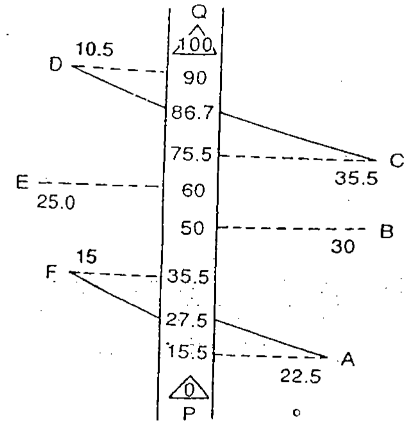


Fig. 7.4

Solution The figure is plotted as shown in Fig. 7.5.

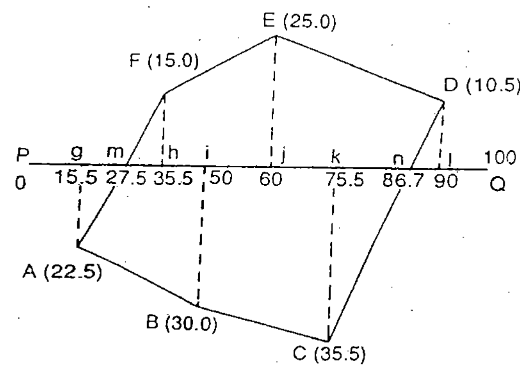


Fig. 7.5

The result is given in the following table:

Sl No.	Figure	Chainage (m)	Base (m)	Offset (m)	Mean offset (m)	Area (m^2) +ve	Area (m^2) − ve	Remark
1	2	3	4	5	6	7	8	9
1	ΔGAM	15.5 & 27.5	12.0	22.5 & 0	11.25	—	135.000	Area = col. 4 × col. 6
2	Trap. GABI	15.5 & 50.0	34.5	22.5 & 30.0	26.25	905.625	—	
3	Trap. IBCK	50.0 & 75.5	25.5	30.0 & 35.5	32.75	835.125	—	

(*Contd.*)

Sl No.	Figure	Chainage (m)	Base (m)	Offset (m)	Mean offset (m)	Area (m²)		Remark
						+ve	– ve	
1	2	3	4	5	6	7	8	9
4	ΔKCN	75.5 & 86.7	11.2	35.5 & 0	17.75	198.800	—	
5	ΔNLD	86.7 & 90	3.3	0 & 10.5	5.25	—	17.325	
6	Trap. LDEJ	60 & 90	30	10.5 & 25.0	17.75	532.500	—	
7	Trap. JEFH	35.5 & 60	24.5	25 & 15	20	490.000	—	
8	ΔFHM	27.5 & 35.5	8.0	15 & 0	7.5	60.000	—	
						3,022.50	152.325	

Required area = 3, 022.050 – 152.325 = 2, 869.725 sq. m

7.4 COMPUTATION OF AREA FROM PLOTTED PLAN

The area may be calculated in the two following ways.

Case I—Considering the entire area The entire area is divided into regions of a convenient shape, and calculated as follows:

(a) By dividing the area into triangles The triangles are so drawn as to equalise the irregular boundary line.

Then the bases and altitudes of the triangles are determined according to the scale to which the plan was drawn. After this, the areas of these triangles are calculated (area = 1/2 × base × altitude). The areas are then added to obtain the total area (Fig. 7.6).

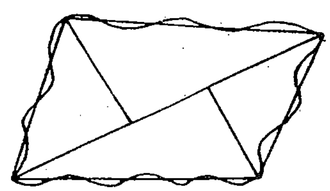


Fig. 7.6

(b) By dividing the area into squares In this method, squares of equal size are ruled out on a piece of tracing paper. Each square represents a unit area, which could be 1 cm² or 1 m². The tracing paper is placed over the plan and the number of full squares are counted. The total area is then calculated by multiplying the number of squares by the unit area of each square (Fig. 7.7).

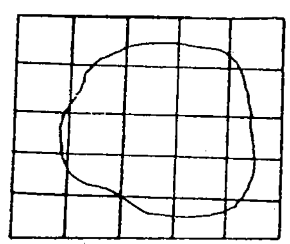


Fig. 7.7

(c) By drawing parallel lines and converting them to rectangles In this method, a series of equidistant parallel lines are drawn on a tracing paper (Fig. 7.8). The constant distance represents a metre or centimetre. The tracing paper is placed

over the plan in such a way that the area is enclosed between the two parallel lines at the top and bottom. Thus the area is divided into a number of strips. The curved ends of the strips are replaced by perpendicular lines (by give and take principle) and a number of rectangles are formed. The sum of the lengths of the rectangles is then calculated. Then,

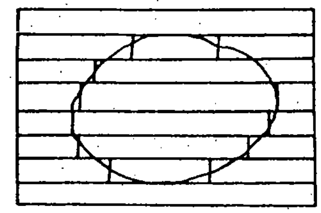


Fig. 7.8

Required area = Σ length of rectangles × constant distance

Case II In this method, a large square or rectangle is formed within the area in the plan. Then ordinates are drawn at regular intervals from the side of the square to the curved boundary. The middle area is calculated in the usual way. The boundary area is calculated according to one of the following rules:

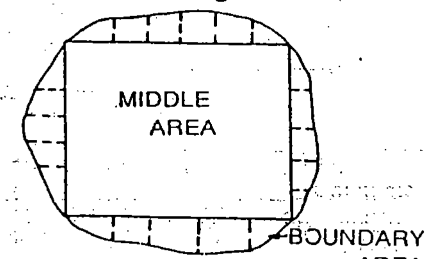


Fig. 7.9

1. The mid-ordinate rule
2. The average ordinate rule
3. The trapezoidal rule
4. Simpson's rule

The various rules are explained in the following sections.

7.5 THE MID-ORDINATE RULE

Consider Fig. 7.10

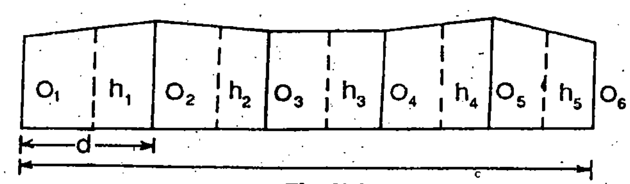


Fig. 7.10

Let $O_1, O_2, O_3, \ldots, O_n$ = ordinates at equal intervals

l = length of base line

d = common distance between ordinates

$h_1, h_2, \ldots h_n$ = mid-ordinates

$$\text{Area of plot} = h_1 \times d + h_2 \times d + \ldots + h_n \times d = d(h_1 + h_2 + \ldots + h_n) \quad (7.1)$$

i.e. Area = common distance × sum of mid-ordinates

7.6 THE AVERAGE-ORDINATE RULE

Refer to Fig. 7.11.

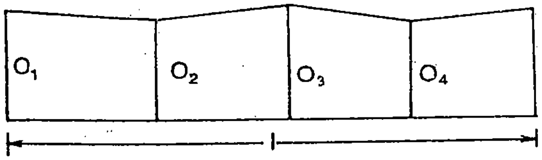


Fig. 7.11

Let $O_1, O_2, \ldots, O_n$ = ordinates or offsets at regular intervals

l = length of base line

n = number of divisions

$n + 1$ = number of ordinates

$$\text{Area} = \frac{O_1 + O_2 + \ldots + O_n}{O_{n+1}} \times l \tag{7.2}$$

i.e.
$$\text{Area} = \frac{\text{sum of ordinates}}{\text{no. of ordinates}} \times \text{length of base line}$$

7.7 THE TRAPEZOIDAL RULE

While applying the trapezoidal rule, boundaries between the ends of ordinates are assumed to be straight. Thus the areas enclosed between the base line and the irregular boundary line are considered as trapezoids.

Consider Fig. 7.12.

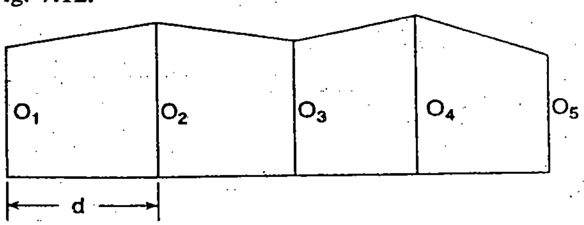


Fig. 7.12

Let $O_1, O_2, \ldots, O_n$ = ordinates at equal intervals

d = common distance

$$\text{1st area} = \frac{O_1 + O_2}{2} \times d$$

$$\text{2nd area} = \frac{O_2 + O_3}{2} \times d$$

$$\text{3rd area} = \frac{O_3 + O_4}{2} \times d$$

....

$$\text{Last area} = \frac{O_{n-1} + O_n}{2} \times d$$

$$\text{Total area} = \frac{d}{2}\{O_1 + 2O_1 + 2O_2 + \ldots + 2O_{n-1} + O_n\} \tag{7.3}$$

$$= \frac{\text{common distance}}{2}\{(\text{1st ordinate} + \text{last ordinate} + 2\ (\text{sum of other ordinate})\}$$

Thus, the trapezoidal rule may be stated as follows:

To the sum of the first and the last ordinate, twice the sum of intermediate ordinates is added. This total sum is multiplied by the common distance. Half of this product is the required area.

Limitation There is no limitation for this rule. This rule can be applied for any number of ordinates.

7.8 SIMPSON'S RULE

In this rule, the boundaries between the ends of ordinates are assumed to form an arc of a parabola. Hence Simpson's rule is sometimes called the parabolic rule.

Refer to Fig. 7.13.

Let

O_1, O_2, O_3 = three consecutive ordinates

d = common distance between the ordinates

Area AFeDC = area of trapezium AFDC + area of segment FeDEF

Simpson's Rule

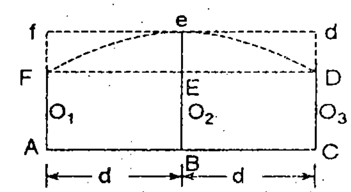


Fig. 7.13

Here,

$$\text{Area of trapezium} = \frac{O_1 + O_3}{2} \times 2d$$

$$\text{Area of segment} = \frac{2}{3} \times \text{area of parallelogram FfdD}$$

$$= \frac{2}{3} \times \text{Ee} \times 2d = \frac{2}{3} \times \left\{O_2 - \frac{O_1 + O_3}{2}\right\} \times 2d$$

So, the area between the first two divisions,

$$\Delta_1 = \frac{O_1 + O_3}{2} \times 2d + \frac{2}{3}\left\{O_2 - \frac{O_1 + O_3}{2}\right\} \times 2d$$

$$= \frac{d}{3}(O_1 + 4O_2 + O_3)$$

Similarly, the area between next two divisions,

$$\Delta_2 = \frac{d}{3}(O_3 + 4O_4 + O_5) \text{ and so on.}$$

$$\therefore \quad \text{Total area} = \frac{d}{3}(O_1 + 4O_2 + 2O_3 + 4O_4 + \ldots + O_n)$$

$$= \frac{d}{3}\{O_1 + O_n + 4(O_2 + O_4 + \ldots) + 2(O_3 + O_5 + \ldots)\}$$

$$= \frac{\text{common distance}}{3}\{\text{1st ordinate + last ordinate}) + 4\text{ (sum of even ordinates)} + 2\text{ (sum of remaining odd ordinates)}\}$$

Thus, the rule may be stated as follows.

To the sum of the first and the last ordinate, four times the sum of even ordinates and twice the sum of the remaining odd ordinates are added. This total sum is multiplied by the common distance. One-third of this product is the required area.

Limitation This rule is applicable only when the number divisions is even, i.e. the number of ordinates is odd.

The trapezoidal rule and Simpson's rule may be compared in the following manner:

Trapezoidal rule	*Simpson's rule*
1. The boundary between the ordinates is considered to be straight.	1. The boundary between the ordinates is considered to be an arc of a parabola.
2. There is no limitation. It can be applied for any number of ordinates.	2. To apply this rule, the number of ordinates must be odd. That is, the number of divisions must be even.
3. It gives an approximate result	3. It gives a more accurate result.

Note Sometimes one, or both, of the end ordinates may be zero. However, they must be taken into account while applying these rules.

7.9 WORKED-OUT PROBLEMS

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

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

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

(a) The mid-ordinate rule

(b) The average-ordinate rule

(c) The trapezoidal rule

(d) Simpson's rule

Solution

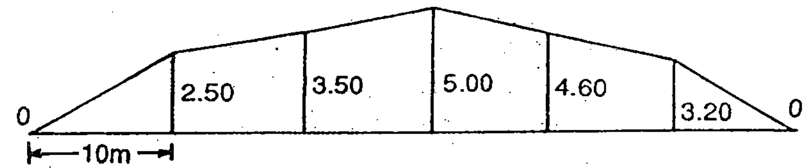


Fig. P.7.1

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

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

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

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

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

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

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

$$\text{Required area} = 10\,(1.25 + 3.00 + 4.25 + 4.80 + 3.90 + 1.60) = 10 \times 18.80 = 188 \text{ m}^2$$

(b) *By average-ordinate rule*:

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

Base length = 10 × 6 = 60 m

Number of ordinates = 7

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

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

(c) *By trapezoidal rule*:

Here $d = 10$

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

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

(d) *By Simpson's rule:*

$d = 10$

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

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

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

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

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

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

(a) The trapezoidal rule
(b) Simpson's rule

Solution (Fig. P-7.2)

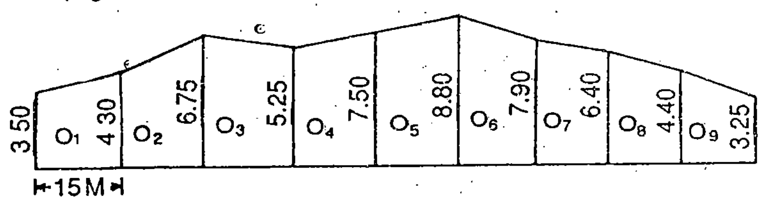


Fig. P-7.2

(a) *By trapezoidal rule:*

$$\text{Required area} = \frac{15}{2}\{3.50 + 3.25 + 2(4.30 + 6.75 + 5.25 + 7.50 + 8.80 + 7.90 + 6.40 + 4.40)\}$$

$$= \frac{15}{2}\{6.75 + 102.60\} = 820.125 \text{ m}^2$$

(b) *Simpson's rule:* If this rule is to be applied, the number of ordinates must be odd. But here the number of ordinate is even (ten).

So, Simpson's rule is applied from O_1 to O_9 and the area between O_9 and O_{10} is found out by the trapezoidal rule.

$$A_1 = \frac{15}{3}\{3.50 + 4.40 + 4(4.30 + 5.25 + 8.80 + 6.40) + 2(6.75 + 7.50 + 7.90)\}$$

$$= \frac{15}{3}\{7.90 + 99.00 + 44.30\} = 756.00 \text{ m}^2$$

$$A_2 = \frac{15}{2}\{4.40 + 3.25\} = 57.38 \text{ m}^2$$

$$\text{Total area} = A_1 + A_2 = 756.00 + 57.38 = 813.38 \text{ m}^2$$

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

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

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

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

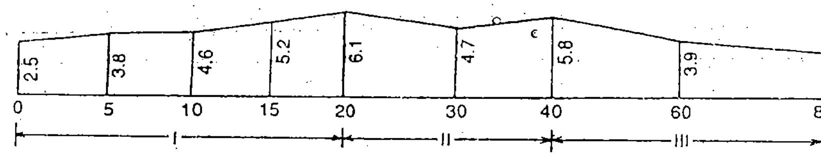


Fig. P.7.3

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

So, the section is divided into three compartments.

Let
Δ_I = area of 1st section
Δ_{II} = area of 2nd section
Δ_{III} = area of 3rd section

Here,
$d_1 = 5$ m
$d_2 = 10$ m
$d_3 = 20$ m

(a) *By Trapezoidal rule:*

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

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

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

$$\text{Total area} = 89.50 + 106.50 + 158.00 = 354.00 \text{ m}^2$$

(b) *By Simpson's rule:*

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

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

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

$$\text{Total area} = 89.66 + 102.33 + 157.33 = 349.32 \text{ m}^2.$$

7.10 COORDINATE METHOD OF FINDING AREA

When offsets are taken at very irregular intervals, then the application of the trapezoidal rule and Simpson's rule is very difficult. In such a câse, the coordinate method is the best.

Procedure From the given distances and offsets, a point is selected as the origin. The coordinated of all other points are arranged with reference to the origin (Fig. 7.14).

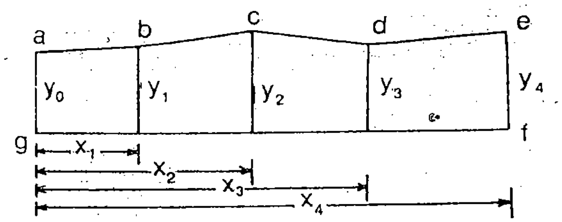


Fig. 7.14

Taking g as the origin, the coordinates of all other points are arranged as follows:

Points	Coordinates	
	X	Y
a	0	y_0
b	x_1	y_1
c	x_2	y_2
d	x_3	y_3
e	x_4	y_4
f	x_4	0
g	0	0
a	0	y_0

The coordinates are arranged in determinant form as follows.

$$\begin{matrix} a & b & c & d & e & f & g & a \\ \frac{y_0}{0} & \frac{y_1}{x_1} & \frac{y_2}{x_2} & \frac{y_3}{x_3} & \frac{y_4}{x_4} & \frac{0}{x_4} & \frac{0}{0} & \frac{y_0}{0} \end{matrix}$$

Sum of products along the solid line,

$$\Sigma P = (y_0 x_1 + y_1 x_2 + \dots + 0.0)$$

Sum of products, along the dotted line

$$\Sigma Q = (0 \cdot y_1 + x_1 y_2 + \dots + 0 \cdot y_0)$$

$$\text{Required area} = 1/2\,(\Sigma P - \Sigma Q)$$

Example The following perpendicular offsets were taken from a chain line to a hedge.

Chainage (m) —0–5.5–12.7–25.5–40.5
Offset (m) —5.25–6.50–4.75–5.20–4.20

Calculate the area between the chain line and the hedge by the coordinate method.

Solution

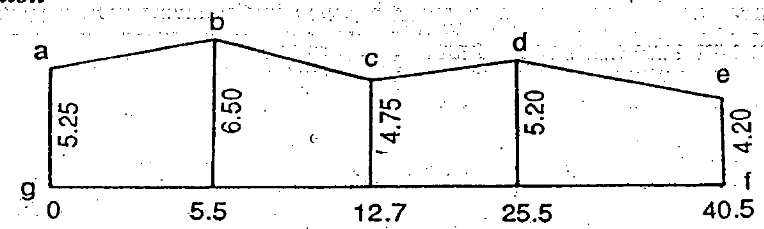


Fig. E.7.1

Taking g as the origin, the coordinates are arranged as follows:

Point	Coordinates	
	X	Y
a	0	5.25
b	5.5	6.50
c	12.7	4.75
d	25.5	5.20
e	40.5	4.20
f	40.5	0
g	0	0
a	0	5.25

Then°the coordinates are arranged in determinant form.

$$\begin{matrix} a & b & c & d & e & f & g & a \\ \frac{5.25}{0} & \frac{6.50}{5.50} & \frac{4.75}{12.7} & \frac{5.20}{25.5} & \frac{4.20}{40.5} & \frac{0}{40.5} & \frac{0}{0} & \frac{5.25}{0} \end{matrix}$$

Sum of products along the solid line,

$$\Sigma P = (5.25 \times 5.50 + 6.50 \times 12.7 + 4.75 \times 25.5 + 5.20 \times 40.5 + 4.20 \times 40.5 + 0 \times 0 + 0 \times 0)$$
$$= 28.88 + 82.55 + 121.13 + 210.60 + 170.10 = 613.26$$

Sum of products along dotted line,

$$\Sigma Q = (0 \times 6.50 + 5.50 \times 4.75 + 12.7 \times 5.20 + 25.5 \times 4.20 + 40.5 \times 0 + 40.5 \times 0 + 0 \times 5.25)$$
$$= 26.13 + 66.04 + 107.10 = 199.27$$

$$\text{Required area} = 1/2\ (\Sigma P - \Sigma Q)$$
$$= 1/2\ (613.26 - 199.27) = 206.995 \text{ m}^2$$

7.11 INSTRUMENTAL METHOD

The instrument used for computation of area from a plotted map is the planimeter. The area obtained by planimeter is more accurate than that obtained by the graphical method. There are various types of planimeter in use. But the Amslar polar planimeter is the most commonly used now. The constructional details of this planimeter are shown in Fig. 7.15.

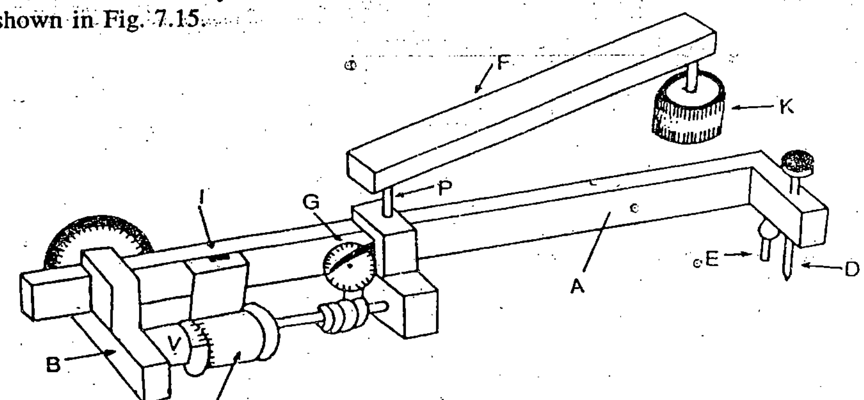


Fig. 7.15

A = Tracing Arm
D = Tracing Point
E = Adjustable support
F = Anchor Point
K= Fulcrum Point
P = Pivot Point
B = Carriage
I = Index Mark
W = Measuring Wheel
V = Vernier Scale
G = Counting Disc

1. It consists of two arms. The arm A is known as the tracing arm. Its length can be adjusted and it is graduated. The tracing arm carries a tracing point D which is moved along the boundary line of the area. There is an adjustable support E which always keeps the tracing point just clear of the surface.
2. The other arm F is known as the pole arm or anchor arm, and carries a needle pointed weight or fulcrum K at one end. The weight forms the centre of rotation. The other end of the pole arm can be pivoted at point P by a ball-and-socket arrangement.
3. There is a carriage B which can be set at various points of the tracing arm with respect to the vernier of the index mark I.

4. The carriage consists of a measuring wheel W and a vernier V. The wheel is divided into 100 divisions and the vernier into 10 divisions. The wheel and the vernier measure readings up to three places of decimal (i.e. 0.125, 0.174, etc.).
5. The wheel is geared to a counting disc which is graduated into 10 divisions. For ten complete revolutions of the wheel, the disc shows a reading of one divisions.

   Thus, the planimeter shows a reading of four digits (i.e. 1.125, 1.174, etc.).

   The counting disc shows—units

   The wheel shows—tenth and hundredth and the vernier shows—thousandth.
6. The planimeter rests on the tracing point, anchor point and the periphery of the wheel.

Table 1 Guiding Tables Supplied by Manufacturer

Scale	Vernier position on tracer bar (i.e. index mark)	Area for one revolution of measuring wheel (*M*)		Constant (*C*)
		Scale	Actual	
1 : 1	21.51	10 sq. inch	10 sq. inch	26.448
3/8″ = 1′ (1 : 32)	30.24	100 sq. ft	14.06 sq. inch	23.617
1/4″ = 1′ (1 : 48)	26.88	200 sq. ft	12.5 sq. inch	24.319
1/2″ = 1′ (1 : 24)	26.88	50 sq. ft	12.5 sq. inch	24.319

Table 2

Scale	Vernier position on tracer bar (i.e. index mark)	Area for one revolution of measuring wheel (*M*)		Constant (*C*)
		Scale	Actual	
1 : 1	33.33	100 cm^2	100 cm^2	23.254
1 : 200	33.33	0.4 m^2	—	23.254
1 : 400	20.83	1 m^2	—	27.133
1 : 500	26.67	2 m^2	—	24.637
1 : 1000	33.33	10 m^2	—	23.547

Procedure of finding the area with a planimeter

1. The vernier of the index mark is set to the exact graduation marked on the tracer arm corresponding to the scale as obtained from the table.

   Suppose the scale is 3/8″ = 1′. Then the vernier of the index mark should be set to 30.24 (obtained from the table).

   If the scale is 1 : 1000, the index mark should be set to 33.33, and so on.
2. The anchor point is fixed firmly in the paper outside or inside the figure. It should be ensured that the tracing point is easily able to reach every point on the boundary line.

But it is always preferable to set the anchor point outside the figure. If the area is very large, it can be divided into a number of sections.

3. A good starting point is marked on the boundary line. A good starting point is one at which the measuring wheel is dead, i.e. a point where the wheel does not revolve even for a small movement of the tracing point.
4. By observing the disc, wheel and vernier, the initial reading (IR) is recorded.
5. The tracing point is moved gently in a clockwise direction along the boundary of the area.
6. The number of times the zero mark of the dial passes the index mark in a clockwise or anticlockwise direction should be observed.
7. Finally, by observing the disc, wheel and vernier the final reading (FR) is recorded.
8. Then, the area of the figure may be obtained from the following expression:

$$\text{Area } A = M\,(FR - IR \pm 10N + C)$$

where

M = Multiplier given in the table
N = Number of times the zero mark of the dial passes the index mark
C = the constant given in the table.
FR = final reading
IR = initial reading

Note:

(a) N is considered to be positive when the zero of the dial passes the index mark in a clockwise direction.
(b) N is considered negative when the zero of the dial passes the index mark in an anticlockwise direction.
(c) The value of C is added only when the anchor point is inside the figure.

Zero circle of the planimeter When the tracing point is moved along a circle without rotation of the wheel (i.e. when the wheel slides without any change in reading), the circle is known as the 'zero circle' or 'circle of correction'. The zero circle is obtained by moving the tracing point in such a way that the tracing arm makes an angle of 90° with the anchor arm (Fig. 7.16).

The anchor point A is the centre of rotation and AT (R′) is known as the radius of the zero circle.

When the anchor point is inside the figure, the area of the zero circle is added to the area computed by planimeter.

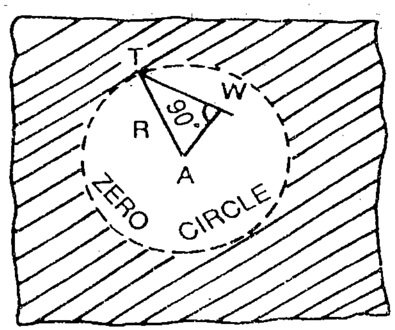


Fig. 7.16

Finding Radius of Zero Circle

1. When the wheel is outside of the pivot point,

A = anchor point
P = pivot point
T = tracing point
W = wheel

Let TP = L
PW = L_1
AP = R
AT = R′ (radius of zero circle)

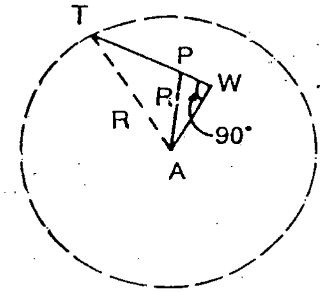


Fig. 7.17

From right-angled triangle AWT (∠AWT = 90°)

$$AT^2 = AW^2 + TW^2$$

or

$$R'^2 = (AP^2 - PW^2) + TW^2 = (R^2 - L_1^2) + (L + L_1)^2$$
$$= R^2 - L_1^2 + L^2 + 2LL_1 + L_1^2 = R^2 + L^2 + 2LL_1$$

$$\therefore \quad R' = \sqrt{R^2 + 2LL_1 + L^2}$$

2. When the wheel is inside of the pivot point,

Let TP = L
PW = L_1
AP = R
AT = R_1
∠AWT = 90°

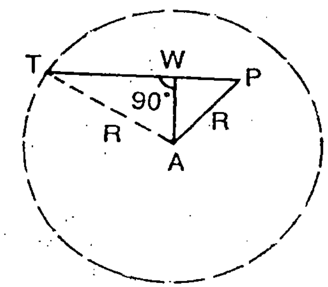


Fig. 7.18

From right-angled triangle AWT,

$$AT^2 = TW^2 + AW^2$$
$$= (TP - PW)^2 + (AP^2 - PW^2)$$
$$R_1^2 = (L - L_1)^2 + (R^2 - L_1^2)$$
$$= L^2 - 2LL_1 + L_1^2 + R^2 - L_1^2 = L^2 - 2LL_1 + R^2$$
$$R_1 = \sqrt{L^2 - 2LL_1 + R^2}$$

Finding Area of Zero Circle

1. By measuring radius of zero circle:
By setting AW and TW at right angles, we can measure the distance AT, which is the radius of the zero circle, R'.

$$\text{Area of zero circle} = \pi R'^2$$

2. When the wheel is placed beyond the pivot point (as shown in Fig. 7.19):
Area of zero circle, $A = \pi R'^2$

$$= \pi (L^2 + 2LL_1 + R^2)$$

where $R' = \sqrt{L^2 + 2LL_1 + R^2}$

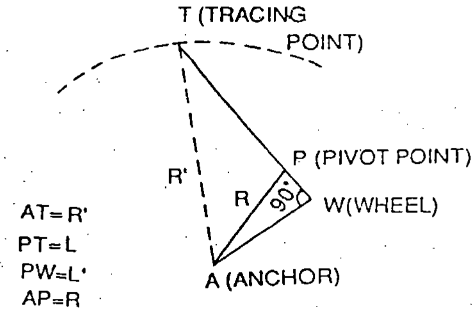


Fig. 7.19

3. When the wheel is placed between the pivot and the tracing point (Fig. 7.20).

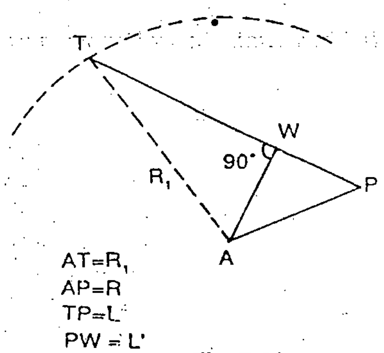


Fig. 7.20

Area of zero circle,

$$A = \pi R_1^2 = \pi (L^2 - 2LL_1 + R^2)$$

where $R_1 = \sqrt{L^2 - 2LL_1 + R^2}$

4. From multiplier and constant:

Area of zero circle, $A = M \times C$

where M = multiplier value given in table
C = constant given in table

5. By planimeter:
A geometrical figure is considered whose actual area is known. After this, the area of the figure is computed by the planimeter.
Then,

Area of zero circle = actual area – area computed by planimeter

Point to be remembered while using the planimeter

1. The map must be placed over a horizontal plane.

2. The anchor point should preferably be kept outside the figure to avoid additive constant.
3. The area of the figure should be measured twice from different starting points.
4. If the area is large, it should be divided into a number of sections; the area of each section may be calculated separately by taking outside poles, and then added to obtain the total area.
5. The initial reading may be set to zero for the sake of simplicity.
6. The tracing point should be moved gently and exactly along the boundary line.
7. The map should not be folded.
8. The surface of the map should be smooth.

7.12 WORKED-OUT PROBLEMS

Problem 1 The following readings were recorded by a planimeter with the anchor point inside the figure:

IR = 9.377, FR = 3.336, M = 100 cm^2, and C = 23.521

Calculate the area of the figure when it is observed that the zero mark of the dial passed the index mark once in the anticlockwise direction.

Solution Given data:

IR = 9.377 M = 100 cm^2
FR = 3.336 C = 23.521
N = – 1 (for anticlockwise rotation)

From the expression, $A = M\,(FR - IR \pm 10N + C)$

we get, $A = 100\,(3.336 - 9.377 - 10 \times 1 + 23.521) = 748$ cm^2

Problem 2 The following particulars were noted while measuring the area of a figure with a planimeter.

(a) IR and FR were 8.652 and 6.798 respectively.
(b) The tracing arm was set to the natural scale.
(c) The zero of the dial passed the index mark once in the anticlockwise direction.
(d) Constant C = 20.
(e) Scale of the map is 1 cm = 10 m.
(f) The anchor point was inside the figure.

Calculate the area of the figure.

Solution Given data

IR = 8.652
FR = 6.798

'Natural scale' means that M = 100 cm^2

$C = 20$

$N = -1$ (for anticlockwise rotation)

Scale: 1 cm = 10 m

Area of the figure

$A = M(FR - IR - 10 \times N + C)$

$= 100 (6.798 - 8.652 - 10 \times 1 + 20 = 814.6$ cm^2

Since the scale is 1 cm = 10 m,

1 cm^2 = 100 m^2

Required area = 814.6 × 100 = 81460 m^2

Problem 3 The area of an irregular figure was measured with a planimeter having the anchor point outside the figure. The initial and final readings were 4.855 and 8.754 respectively. The tracing arm was set to the natural scale. The scale of the map was 1 cm = 5 m. Find the area of the figure.

Solution Here, given data:

IR = 4.855

FR = 8.754

M = 100 cm^2 (natural scale)

N = 0 (because there is no comment about the crossing of the index mark

Scale: 1 cm = 5 m

C = 0 (anchor point outside)

So,

Area $A = M$ (FR − IR)

= 100 (8.754 − 4.855)

= 389.9 cm^2

Required area = 389.9 × 25 = 9,747.5 m^2 (∵ 1 cm^2 = 25 m^2)

Problem 4 A field of area 0.16 hectare was plotted to a scale of 1 cm = 4 m. The area of this plan was measured with a certain setting of a planimeter, and the following details were obtained, when the anchor point was outside the area:

IR = 3.415, FR = 4.415, $N = 0$

Another area was traced with the same setting of the planimeter by keeping the anchor point inside the area, and the following observations were obtained:

FR − IR = 2.250 and $N = -1$

The value of the additive constant for this scale was noted as 21.22. Find the area in the second case, and the area of the zero circle.

Solution *In first case*

Given data:

IR = 3.415, FR = 4.115, $N = 0$, M = ?

$C = 0$ (anchor point outside)

Scale: 1 cm = 4 m

∴ 1 cm^2 = 16 m^2

Area on the ground = 0.16 × 10,000 (1 hectare = 10,000 m^2)

= 1,600 m^2

$$\text{Area of map} = \frac{1,600}{16} = 100 \text{ cm}^2$$

So, A = 100 cm^2 (measured area on plan)

From the relation

$A = M$ (FR − IR)

$100 = M (4.415 - 3.415) = M \times 1$

$M = 100$

In second case

Given data: FR − IR = 2.250

$C = 21.22$

$N = -1$

Required area, $A = M$ (FR − IR − 10 × 1 + 21.22)

= 100 (2.250 − 10 + 21.22) = 1, 347 cm^2

Area of zero circle = $M \times C$

= 100 × 21.22 = 2,122 cm^2

Problem 5 The following readings were obtained while describing the perimeter of a rectangle of size 15 cm × 10 cm with the anchor point inside the rectangle and tracing arm set to the natural scale (M = 100): IR = 0.686 and FR = 9.976. The zero of the counting disc passed the index mark twice in the anticlockwise direction. Find the area of the zero circle.

Solution Area of rectangle = 15 × 10 = 150 cm^2

IR = 0.686, FR = 9.976, $N = -2$, $M = 100$

C = ?

Measured area of rectangle = M (FR − IR − 10N + C)

= 100 (9.976 − 0.686 − 10 × 2 + C)

= 100 (− 10.710 + C) (1)

Again, measured area = 150 cm^2 (2)

From (1) and (2),

100 (− 10.710 + C) = 150

C = 1.5 + 10.710 = 12.21

Hence, area of zero circle = $M \times C$

= 100 × 12.21 = 1,221 cm^2

Problem 6 Calculate the area of the zero circle with the following data:

IR	FR	Position of anchor point	Remark
7.775	4.825	Outside the figure	The zero crosses the index mark once clockwise
2.325	8.755	Inside the figure	The zero crosses the index mark twice anticlockwise

Solution *In first case:*

$N = 1$
$C = 0$ (as anchor point is outside the figure)
$M = 100$

$$\text{Area of figure} = M\,(\text{FR} - \text{IR} + 10\,N) = 100\,(4.825 - 7.775 + 10 \times 1) = 100 \times 7.050 = 705.0 \text{ cm}^2 \quad (1)$$

In second case:

$M = 100$
$N = -2$
$C = ?$

$$\text{Area of figure} = M\,(\text{FR} - \text{IR} - 10\,N + C) = 100\,(8.755 - 2.325 - 10 \times 2 + C) = 100\,(-13.570 + C) \quad (2)$$

Equating (1) and (2),

$$100\,(-13.570 + C) = 705.0$$
$$C = 7.050 + 13.570 = 20.620$$

Hence, Area of zero circle $= M \times C = 100 \times 20.620 = 2062 \text{ cm}^2$

SHORT QUESTIONS WITH ANSWERS FOR VIVA

Q. 1 State the trapezoidal rule. What are the considerations and limitations of this rule?

Ans. "To the sum of the first and the last ordinate, twice the sum of the intermediate ordinates is added. This total sum is multiplied by the common distance. Half of this product is the required area". This is the trapezoidal rule.

The boundaries between the ends of ordinates are assumed to be straight lines. There is no limitation in this rule. It can be applied at any number of ordinates.

Q. 2 State Simpson's rule. What are the considerations and limitations of this rule.

Ans. To the sum of the first and the last ordinate, four times the sum of even ordinates and twice the sum of odd ordinates are added. This total sum is multiplied by the common distance. One-third of this product is the required area." This is Simpson's rule.

The boundary between the ordinates is assumed to form an arc of a parabola. To apply this rule, the number of ordinates must be odd.

Q. 3 What is a planimeter?

Ans. It is an instrument for measuring the area of a field from the map.

Q. 4 What is a zero circle?

Ans. When a circle is described by the tracing point without a change in reading in the measuring wheel, then that circle is known as the zero circle.

Q. 5 Give the simplest method for finding the area of a zero circle from the manufacturer's table.

Ans. Area of zero circle $= M \times C$

where, M = Multiplier
C = Constant

The values of both M and C are available in the table.

Q. 6 What is the need of finding the area of the zero circle?

Ans. When the anchor point is inside the figure, the computed area does not cover the whole area. It is less by the area of the zero circle. In that case, the area of the zero circle is added to the computed area to obtain the actual area.

EXERCISES

1. (a) State the trapezoidal rule and Simpson's rule. What is the limitation of Simpson's rule.

   (b) The following perpendicular offsets were taken from a chain line to a hedge:

Distance (m)—	0	6	12	18	24	30	36
Offset (m)—	5.40	4.50	3.60	2.70	1.80	2.25	3.15

   Calculate the area enclosed between the chain line and the offsets by (i) trapezoidal rule, and (ii) Simpson's rule. (Ans. 114.75 m^2, 114.3 m^2)

2. The following perpendicular offsets were taken from a chain line to a hedge:

Distance (m)—	0	5	10	15	20	30	40	50	65	80
Offset (m)—	3.40	4.25	2.60	3.70	2.90	1.80	3.20	4.50	3.70	2.80

   Calculate the area by: (i) Trapezoidal rule, and (ii) Simpson's rule (Ans. 261.75 m^2, 265.5 m^2)

3. The following perpendicular offsets were taken at 10 m intervals from a chain line to an irregular boundary line:

   3.10, 4.20, 5.35, 6.45, 7.15, 8.25, 7.95 and 5.20 m

   Find the area: (i) trapezoidal rule, and (ii) Simpson's rule. (Ans. 436.50 m^2, 438.95 m^2)

4. The following perpendicular offsets were taken from a chain line to an irregular boundary line:

   Chainage (m)—0–6.5–16.2–27.2–39.6
   Offset (m)—3.50–4.75–5.20–6.30–7.36

   Calculate the area between the chain line and the boundary. (Ans. 223.012 m^2)

5. Calculate the area of the figure corresponding to the following data recorded by planimeter:

   (a) IR = 2.436 (b) FR = 7.745
   (c) M = 100 cm^2 (d) C = 20.00
   (e) The figure traversed clockwise with the anchor point inside and the zero of the dial passed the index mark once in the reverse direction.

   (Ans. 1530.9 cm^2)

6. Select the correct alternative for questions (i) through (viii).
   (i) In the trapezoidal formula, the line joining the top of the ordinates is assumed to be
   (a) Curved (b) Straight (c) Circular
   (ii) In Simpson's formula, the line joining the top of the ordinates is considered
   (a) Parabolic (b) Elliptical (c) Circular
   (iii) In Simpson's formula, the number of ordinates must be
   (a) Odd (b) Even (c) Either odd or even
   (iv) In the trapezoidal formula, the number of divisions should be
   (a) Even (b) Odd (c) Either odd or even
   (v) Irregular area may be computed by an instrument known as the
   (a) Pentagraph (b) Planimeter (c) Passometer
   (vi) When the tracing point is moved along a circle without rotation of the wheel, then the circle is known as the
   (a) Zero circle (b) Ortho circle (c) Circum circle
   (vii) When the anchor point is inside the figure, the area of the zero circle is
   (a) Added (b) Subtracted (c) Multiplied
   (viii) The value of the planimeter constant (*C*) is added only when
   (a) The anchor point is inside the figure
   (b) The anchor point is outside the figure
   (c) The anchor point is just on the boundary line.

ANSWERS

6. (i) b (ii) a (iii) a (iv) c (v) b
   (vi) a (vii) a (viii) a

8

Computation of Volume

8.1 INTRODUCTION

For computation of the volume of earth work, the sectional areas of the cross-section which are taken transverse to the longitudinal section during profile levelling are first calculated. Again, the cross-sections may be of different types, namely: (i) level (ii) two-level, (iii) three-level, (iv) side-hill two-level, and (v) multi-level.

The methods of calculating the areas of such sections are discussed in Sec. 8.2.

After calculation of cross-sectional areas, the volume of earth work is calculated by: (i) the trapezoidal (or average end area) rule, and (ii) the prismoidal rule.

Notes: 1. The prismoidal rule gives the correct volume directly.
2. The trapezoidal rule does not give the correct volume. Prismoidal correction should be applied for this purpose. This correction is always subtractive.
3. Cutting is denoted by a positive sign and filling by a negative sign.

8.2 FORMULAE FOR CALCULATION OF CROSS-SECTIONAL AREA

A. Level Section

When the ground is level along the transverse direction:

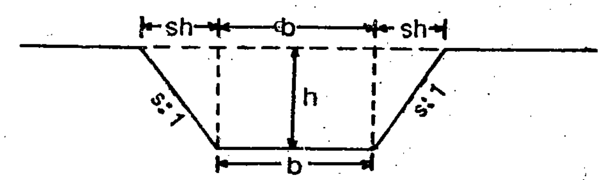


Fig. 8.1

$$\text{Cross-sectional area} = \frac{b + b + 2sh}{2} \times h$$
$$= (b + sh)h \qquad (1)$$

Example Calculate the sectional area of an embankment 10 m wide, with a side slope of 2 : 1. The ground is level in a transverse direction to the centre line. The central height of the embankment is 2.5 m.

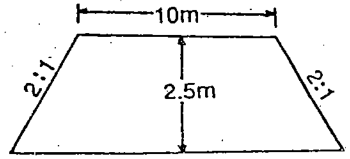


Fig. 8.2

Solution

Here $b = 10$ m

$S = 2$

$\therefore h = 2.5$ m

From Eq. 1,

$\therefore$ Cross-sectional area $= (b + sh)h$

$= (10 + 2 \times 2.5) \times 2.5 = 37.5 \text{ m}^2$

B. Two-Level Section

When the ground surface has a transverse slope:

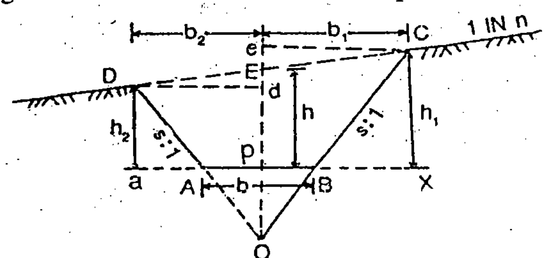


Fig. 8.3

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

$$Bx = sh_1$$

$$b_1 = \frac{b}{2} + sh_1 \qquad \text{(a)}$$

$$Ee = (h_1 - h)$$

and $$b_1 = n \times Ee = n(h_1 - h) \qquad \text{(b)}$$

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

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

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

From (2), and (a)

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

Similarly,

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

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

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

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

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

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

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

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

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

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

Solution Here, $b = 10$ m $\quad s = 1$

$n = 6 \quad h = 3$ m

From Eq. (3),

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

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

From Eq. (5),

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

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

From Eq. (6),

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

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

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

C. Three-level Section

When the transverse slope is not uniform:

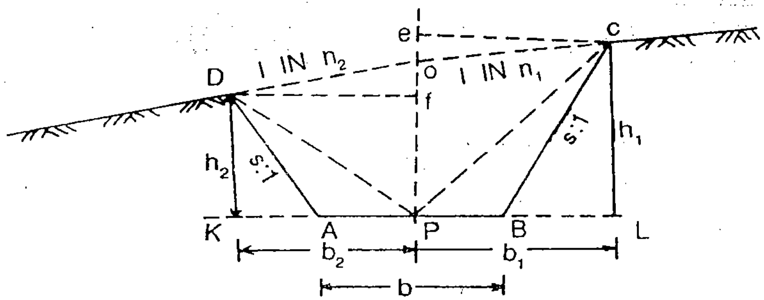


Fig. 8.4

$$\text{Area ABCOD} = \Delta DOP + \Delta COP + \Delta DAP + \Delta BCP$$

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

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

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

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

Deduction of formula for b_2 and b_1

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

Also

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

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

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

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

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

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

Similarly,

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

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

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

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

Solution From Eq. (7), we know that

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

Data for cross-section at station 1:

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

Cross-sectional area at station 1:

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

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

Data for cross-section at station 2:

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

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

Cross-sectional area at station 2:

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

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

D. Side-Hill Two-Level Section

When the ground surface has a transverse slope, but the slope of the ground cuts the formation level partly in cutting and partly in filling, the following method is adopted.

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

which is similar to Eq. 2.

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

which is similar to Eq. (3),

Then, h_2 and b_2 are deduced as follows:

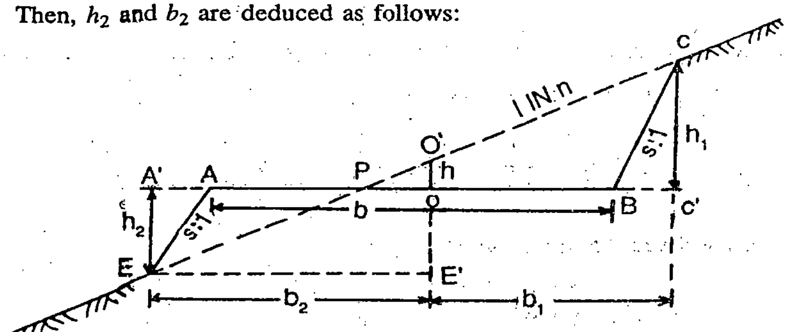


Fig. 8.5

From the accompanying figure,

$$b_2 = \frac{b}{2} + \text{AA}' = \frac{b}{2} + sh_2 \quad \text{(a)}$$

Again $$b_2 = \text{EE}' = \text{O'E}' \times n = (h + h_2)n \quad \text{(b)}$$

From (a) and (b), $$\frac{b}{2} + sh_2 = (h + h_2)n$$

or $$h_2(n - s) = \frac{b}{2} - hn = n\left(\frac{b}{2n} - h\right)$$

$\therefore$ $$h_2 = \frac{n}{(n-s)} \times \left(\frac{b}{2n} - h\right) \quad (12)$$

So, from (a), $$b_2 = \frac{b}{2} + \frac{ns}{n-s} \times \left(\frac{b}{2n} - h\right) \quad (13)$$

Area in cutting:
Area of ΔPBC,

$$A_1 = \frac{1}{2} \times \text{PB} \times h_1$$

Here, $$\text{PB} = \text{OB} + \text{OP} = \frac{b}{2} + nh$$

$$A_1 = \frac{1}{2} \times \left(\frac{b}{2} + nh\right)\left\{\frac{n}{n-s} \times \left(\frac{b}{2n} + h\right)\right\}$$

$\therefore$ $$= \frac{1}{2}\left(\frac{b}{2} + nh\right)\frac{1}{n-s} \times \left(\frac{b}{2} + r\,h\right)$$

$$= \frac{1}{2}\left[\frac{\{(b/2) + nh\}^2}{n-s}\right] \quad (14)$$

Area in filling:
Area of ΔPAE

$$A_2 = \frac{1}{2}\text{PA} \times h_2$$

Here, $$\text{PA} = \frac{b}{2} - nh$$

$\therefore$ $$A_2 = \frac{1}{2}\left(\frac{b}{2} - nh\right)\left\{\frac{n}{n-s} \times \left(\frac{b}{2n} - h\right)\right\}$$

$$= \frac{1}{2}\left(\frac{b}{2} - nh\right) \times \frac{1}{n-s} \times \left(\frac{b}{2} - nh\right)$$

$$= \frac{1}{2}\left[\frac{\{(b/2) - nh\}^2}{n-s}\right] \quad (15)$$

In the above case, the side slopes for cutting and filling are assumed to be equal. But in actual practice, the side slope of cutting is different from that of filling. Let the side slope of cutting be $s_1 : 1$.

Then,

$$b_1 = \frac{b}{2} + \frac{ns_1}{n - s_1} \times \left(h + \frac{b}{2n}\right) \quad (16)$$

Area in cutting,

$$A_1 = \frac{1}{2}\left\{\frac{\{(b/2) + nh\}^2}{n - s_1}\right\} \tag{17}$$

Example The width at formation of a certain road is 10 m and the side slopes are of 1 : 1 in cutting and 2 : 1 in filling. The original ground has a slope of 1 in 5 (fall). If the depth of excavation at the centre is 0.8 m, find the areas of cutting and filling.

Solution *(a) Area of cutting:*

$h = 0.8$ m $n = 5$
$s = 1$ $b = 10$ m

From Eq. (14),

$$\text{Required area} = \frac{1}{2}\left[\frac{\{(b/2) + nh\}^2}{n - s}\right]$$

$$= \frac{1}{2}\left[\frac{\{(10/2) + 5 \times 0.8\}^2}{5 - 1}\right] = \frac{1}{2}\left\{\frac{(5 + 4)^2}{4}\right\} = 10.1\ \text{m}^2$$

(b) Area of filling:

$h = 0.8$ m
$n = 5$
$s = 2$
$b = 10$ m

From Eq. (15),

$$\text{Required area} = \frac{1}{2}\left\{\frac{[(b/2) - nh]^2}{n - s}\right\}$$

$$= \frac{1}{2}\left\{\frac{[(10/2) - 5 \times 0.8]^2}{5 - 2}\right\}$$

$$= \frac{1}{2}\left\{\frac{(5 - 4.0)^2}{3}\right\}$$

$$= 0.16\ \text{m}^2$$

E. Multi-Level Section

The cross-sectional data pertaining to an irregular section are noted in the following form:

Left		Centre	Right	
$\frac{\pm h_2}{b_2}$	$\frac{\pm h_1}{b_1}$	$\frac{\pm h}{0}$	$\frac{\pm h_3}{b_3}$	$\frac{\pm h_4}{b_4}$

A positive sign in the numerator denotes a cut, and a negative sign indicates a fill.

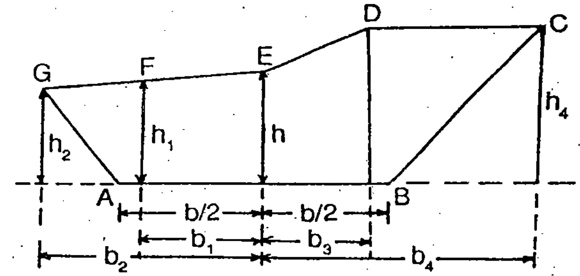


Fig. 8.6

The denominator denotes corresponding horizontal distance from the centre. Starting from the centre (E) and running outwards to the right and left, the coordinates of the vertices are arranged, irrespective of algebraic sign, in determinant form:

$$\begin{matrix} A & G & F & E & D & C & B \\ \frac{0}{b/2} & \frac{h_2}{b_2} & \frac{h_1}{b_1} & \frac{h}{0} & \frac{h_3}{b_3} & \frac{h_4}{b_4} & \frac{0}{b/2} \end{matrix}$$

The sum of the products of the coordinates joined by solid lines is given by

$$\Sigma P = h_3 \times 0 + h_4 \times b_3 + 0 \times b_4 + h_1 \times 0 + h_2 \times b_1 + 0 \times b_2$$

The sum of the products of the coordinates joined by dotted lines is given by

$$\Sigma Q = h \times b_3 + h_3 \times b_4 + h_4 \times \frac{b}{2} + h \times b_1 + h_1 \times b_1 + h_2 \times \frac{b}{2}$$

$$\text{Area} = \frac{1}{2}(\Sigma P \sim \Sigma Q)$$

Example The following are the data corresponding to an irregular cross-section. The width of the road at formation level is 6 m. The side slope is 1 : 1. Calculate the cross-sectional area.

Left		Centre	Right	
$\frac{+2.25}{5.50}$	$\frac{+3.20}{3.00}$	$\frac{+3.75}{0.00}$	$\frac{+6.20}{4.50}$	$\frac{+7.0}{9.0}$

Starting from the centre (D) and running outwards to the right and left, the coordinates of the vertices are arranged irrespective of sign in determinant form as follows:

$$\begin{matrix} A & B & C & D & E & F & G \\ \frac{0}{3.0} & \frac{2.25}{5.50} & \frac{3.20}{3.0} & \frac{3.75}{0} & \frac{6.20}{4.50} & \frac{7.0}{9.0} & \frac{0}{3.0} \end{matrix}$$

Sum of the products of the coordinates joined by firm lines is given by

$$\Sigma P = 6.20 \times 0 + 7.0 \times 4.5 + 0 \times 9.0 + 3.20 \times 0 + 2.25 \times 3.0 + 0 \times 5.5$$
$$= 0 + 31.50 + 0 + 0 + 6.75 + 0$$
$$= 38.25$$

Sum of the products of the coordinates joined by dotted lines is given by

$$\Sigma Q = 3.75 \times 4.5 + 6.2 \times 9.0 + 7.0 \times 3.0 + 3.75 \times 3.0 + 3.2 \times 5.5 + 2.25 \times 3.0$$
$$= 16.87 + 55.80 + 21.00 + 11.25 + 17.60 + 6.75$$
$$= 129.27$$

$$\text{Area} = \frac{1}{2}(\Sigma P - \Sigma Q)$$
$$= \frac{1}{2}(38.25 - 129.27)$$
$$= 45.51 \text{ m}^2 \quad \text{[negative sign has no significance]}$$

8.3 FORMULA FOR CALCULATION OF VOLUME

D = common distance between sections

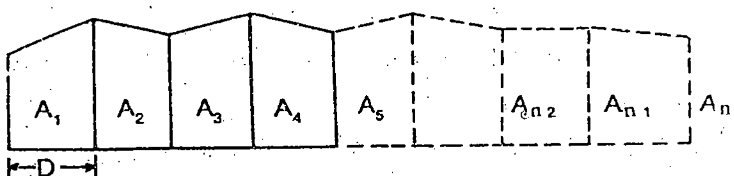


Fig. 8.7

A. Trapezoidal Rule (Average End Area Rule)

Volume (cutting or filling), $V = \frac{D}{2}\{A_1 + A_n + 2(A_2 + A_3 + \ldots + A_{n-1})\}$

i.e. volume = $\frac{\text{common distance}}{2}$ {area of 1st section + area of last section + 2 (sum of area of other sections)}

B. Prismoidal Formula

Volume (cutting or filling), $V = \frac{D}{3}\{A_1 + A_n + 4(A_2 + A_4 + A_{n-1}) + 2(A_3 + A_5 + \ldots + A_{n-2})\}$

i.e., $V = \frac{\text{common distance}}{3}$ {Area of 1st section + area of last section + 4 (sum of areas of even sections) + 2 (sum of areas of odd sections)}

Note: The prismoidal formula is applicable when there are an odd number of sections. If the number of sections is even, the end strip is treated separately and the area is calculated according to the trapezoidal rule. The volume of the remaining strips is calculated in the usual manner by the prismoidal formula. Then both the results are added to obtain the total volume.

8.4 PRISMOIDAL CORRECTION FOR TRAPEZOIDAL OR AVERAGE END AREA RULE

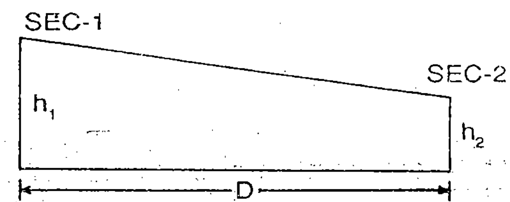


Fig. 8.8

1. Prismoidal correction for level section:

$$C_p = \frac{D \times s}{6}(h_1 - h_2)^2$$ (Considering, transverse slope = 1 in n side slope = s : 1)

2. Prismoidal correction for two-level section:

$$C_p = \frac{D \times s}{6} \times \left(\frac{n^2}{n^2 - s^2}\right) \times (h_1 - h_2)^2$$

3. Prismoidal correction for side hill two-level section:

(a) C_p (for cutting) $= \frac{D}{12(n - s_1)} \times n^2(h_1 - h_1)^2$ (side slope = s_1 : 1)

(b) C_p (for filling) $= \frac{D}{12(n - s_2)} \times n^2(h_1 - h_2)^2$ (side slope = s_2 : 1)

4. Prismoidal correction for three-level section:

$$C_p = \frac{D}{12}(h_1 - h_2) \text{ (whole width of 1st section} - \text{whole width of 2nd section)}$$

8.5 WORKED-OUT PROBLEMS

Problem 1 An embankment of width 10 m and side slopes 1 ½ : 1 is required to be made on a ground which is level in a direction transverse to the centre line. The central heights at 40 m intervals are as follows:

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

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

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

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

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

(a) Volume according to trapezoidal formula:

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

(b) Volume calculated in prismoidal formula:

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

Problem 2 A railway embankment of formation width of 8 m and side slope 2 : 1 is to be constructed. The ground level along the centre line is as follows:

Chainage—	0	50	100	150	200	250
GL (m)—	115.75	114.35	116.80	115.20	118.50	118.25

The embankment has a rising gradient of 1 in 100, and the formation level at zero chainage is 115.00. Assuming the ground is level across the centre line, compute the volume of earth woık.

Solution Rise per 50 m $= \frac{50}{100} = 0.50$ m

Chainage	GL	FL	Cutting (+)	Filling (−)	Sec.
0	115.75	115.00	0.75		A_1
50	114.35	115.50		1.15	A_2
100	116.80	116.00	0.80		A_3
150	115.20	116.50		1.30	A_4
200	118.50	117.00	1.50		A_5
250	118.25	117.50	0.75		A_6

(a) $$\frac{x_1}{0.75} = \frac{50 - x_1}{1.15}$$

or $$1.15x_1 = 37.5 - 0.75x_1$$

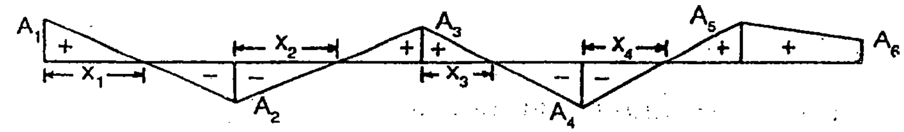


Fig. P.8.1

$$x_1 = 19.74 \text{ m}$$

(b) $$\frac{x_2}{1.15} = \frac{50 - x_2}{0.80}$$

or $$0.80x_2 = 57.5 - 1.15x_2$$
$$x_2 = 29.44 \text{ m}$$

(c) $$\frac{x_3}{0.80} = \frac{50 - x_3}{1.30}$$

or $$1.30x_3 = 40 - 0.80x_3$$
$$x_3 = 19.05 \text{ m}$$

(d) $$\frac{x_4}{1.30} = \frac{50 - x_4}{1.50}$$

or $$1.50x_4 = 65 - 1.30x_4$$
$$x_4 = 23.21 \text{ m}$$

Calculation of area, by the formula: $A = (b + sh)h$—

$A_1 = (8 + 2 \times 0.75) \times 0.75 = 7.13 \text{ m}^2$
$A_2 = (8 + 2 \times 1.15) \times 1.15 = 11.85 \text{ m}^2$
$A_3 = (8 + 2 \times 0.80) \times 0.80 = 7.68 \text{ m}^2$
$A_4 = (8 + 2 \times 1.30) \times 1.30 = 13.78 \text{ m}^2$
$A_5 = (8 + 2 \times 1.50) \times 1.50 = 16.50 \text{ m}^2$
$A_6 = (8 + 2 \times 0.75) \times 0.75 = 7.13 \text{ m}^2$

Calculation of volume

(a) *From chainage 0 to 50:*

$$\text{Cutting} = \frac{7.13 + 0}{2} \times 19.74 = 70.37 \text{ m}^3$$

$$\text{Filling} = \frac{0 + 11.85}{2} \times 30.26 = 179.29 \text{ m}^3$$

(b) *From chainage 50 to 100:*

$$\text{Filling} = \frac{11.85 + 0}{2} \times 29.49 = 174.73 \text{ m}^3$$

$$\text{Cutting} = \frac{0 + 7.68}{2} \times 20.51 = 78.76 \text{ m}^3$$

(c) *From chainage 100 to 150:*

$$\text{Cutting} = \frac{7.68 + 0}{2} \times 19.05 = 73.15 \text{ m}^3$$

$$\text{Filling} = \frac{0 + 13.78}{2} \times 30.95 = 213.25 \text{ m}^3$$

(d) *From chainage 150 to 200:*

$$\text{Filling} = \frac{13.78 + 0}{2} \times 23.2 = 159.92 \text{ m}^3$$

$$\text{Cutting} = \frac{0 + 16.50}{2} \times 26.8 = 221.02 \text{ m}^3$$

(e) *From chainage 200 to 250:*

$$\text{Cutting} = \frac{16.50 + 7.13}{2} \times 50 = 590.75 \text{ m}^3$$

$$\text{Total cutting} = 70.37 + 78.76 + 73.15 + 221.02 + 590.75 = 1034.05 \text{ m}^3$$

$$\text{Total filling} = 179.29 + 174.73 + 213.55 + 159.92 = 727.19 \text{ m}^3$$

Problem 3 The ground level along the centre line of a road is given below.

Chainage (m)—	0	50	100	150	200	250	300
GL (m)—	117.50	116.25	115.95	116.65	117.20	117.85	115.75

It is proposed that the formation level of RL 115.00 should be kept constant of starting from the chainage 'zero'. The formation width of the road is 8 m and the side slope 1 : 1. The ground is level transverse to the centre line.

Solution

Chainage	GL	FL	Cutting
0	117.50	115.00	2.50
50	116.25	115.00	1.25
100	115.95	115.00	0.95
150	116.65	115.00	1.65
200	117.20	115.00	2.20
250	117.85	115.00	2.85
300	115.75	115.00	0.75

Area is calculated according to the equation,

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

where

$$b = 8 \text{ m}$$
$$s = 1$$

and

$$A_1 = (8 + 1 \times 2.50) \times 2.50 = 26.25 \text{ m}^2$$
$$A_2 = (8 + 1 \times 1.25) \times 1.25 = 11.56 \text{ m}^2$$

$$A_3 = (8 + 1 \times 0.95) \times 0.95 = 8.50 \text{ m}^2$$
$$A_4 = (8 + 1 \times 1.65) \times 1.65 = 15.92 \text{ m}^2$$
$$A_5 = (8 + 1 \times 2.20) \times 2.20 = 22.44 \text{ m}^2$$
$$A_6 = (8 + 1 \times 2.85) \times 2.85 = 30.92 \text{ m}^2$$
$$A_7 = (8 + 1 \times 0.75) \times 0.75 = 6.56 \text{ m}^2$$

Calculation of volume by prismoidal formula

$$\text{Volume} = \frac{50}{3} \{26.25 + 6.56 + 4(11.56 + 15.92 + 30.92) + 2(8.50 + 22.44)\}$$
$$= 5{,}471.5 \text{ m}^3$$

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

Contour (m)	270	275	280	285	290
Area (m²)	2,050	8,400	16,300	24,600	31,500

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

Solution (a) Volume according to trapezoidal formula

$$= \frac{5}{2} \{2,050 + 31,500 + 2(8,400 + 16,300 + 24,600)\}$$
$$= 330,375 \text{ m}^3$$

(b) Volume by prismoidal formula:

$$= \frac{5}{3} \{2,050 + 31,500 + 4(8,400 + 24,600) + 2(16,300)\}$$
$$= 330,250 \text{ m}^3$$

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

Solution

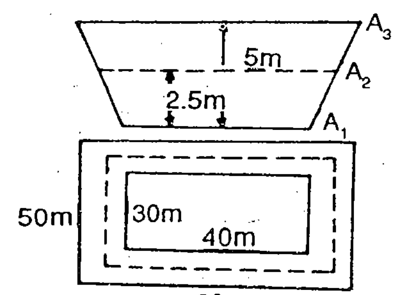


Fig. P.8.2

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

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

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

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

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

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

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

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

$$\text{Volume according to prismoidal formula} = \frac{2.5}{3}\{1{,}200 + 3{,}000 + 4\,(2{,}000)\}$$
$$= 10{,}166.66 \text{ m}^3$$

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

Solution

First section:

$$b = 8 \text{ m}, \quad h = 2\text{m}, \quad n = 10, \quad s = 1$$

From Eq. (3),

$$b_1 = \frac{b}{2} + \frac{n \times s}{n - s}\left(h + \frac{b}{2n}\right) = \frac{8}{2} + \frac{10 \times 1}{10 - 1} \times \left(2 + \frac{8}{2 \times 10}\right) = 6.67 \text{ m}$$

From Eq. (5),

$$b_2 = \frac{b}{2} + \frac{ns}{n + s}\left(h - \frac{h}{2h}\right) = \frac{8}{2} + \frac{10 \times 1}{10 + 1} \times \left(2 - \frac{8}{2 \times 10}\right) = 5.45 \text{ m}$$

From Eq. (6),

$$\text{Area } A_1 = \frac{1}{2}\left\{\left(\frac{b}{2s} + h\right)(b_1 + b_2) - \frac{b^2}{2s}\right\}$$
$$= \frac{1}{2}\left\{\left(\frac{8}{2 \times 1} + 2\right)(6.67 + 5.45) - \frac{8^2}{2 \times 1}\right\} = 20.36 \text{ m}^2$$

Second section:

$$b = 8 \text{ m}, h = 3 \text{ m}, n = 10, s = 1$$

$$b_1 = \frac{8}{2} + \frac{10 \times 1}{10 - 1}\left(3 + \frac{8}{2 \times 10}\right) = 7.78 \text{ m}$$

$$b_2 = \frac{8}{2} + \frac{10 \times 1}{10 + 1}\left(3 - \frac{8}{2 \times 10}\right) = 6.36 \text{ m}$$

$$\text{Area } A_2 = \frac{1}{2}\left\{\left(\frac{8}{2 \times 1} + 3\right)(7.88 + 6.36) - \frac{8^2}{2 \times 1}\right\} = 33.49 \text{ m}^2$$

Third section:

$$b = 8 \text{ m}, h = 4 \text{ m}, n = 10, s = 1$$

$$b_1 = \frac{8}{2} + \frac{10 \times 1}{10 + 1}\left(4 + \frac{8}{2 \times 10}\right) = 8.88 \text{ m}$$

$$b_2 = \frac{8}{2} + \frac{10 \times 1}{10 + 1}\left(4 - \frac{8}{2 \times 10}\right) = 7.27 \text{ m}$$

$$\text{Area } A_3 = \frac{1}{2}\left\{\left(\frac{8}{2 \times 1} + 4\right)(8.88 + 7.27) - \frac{8^2}{2 \times 1}\right\} = 48.64 \text{ m}^2$$

Volume by prismoidal formula,

$$V = \frac{40}{3}\{20.36 + 48.64 + 4 \times (33.49)\} = 2{,}706.13 \text{ m}^3$$

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

Formation width = 10 m

Side lope = 1 : 1

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

Solution

First section:

$$b = 10, h = 1.00 \text{ m}, n = 10, s = 1$$

From Eq. (3),

$$b_1 = \frac{10}{2} + \frac{10 \times 1}{10 - 1} \times \left(1 + \frac{10}{2 \times 10}\right) = 6.66 \text{ m}$$

From Eq. (5)

$$b_2 = \frac{10}{2} + \frac{10 \times 1}{10 + 1} \times \left(1 - \frac{10}{2 \times 10}\right) = 5.45 \text{ m}$$

From Eq. (6),

$$\text{Area } A_1 = \frac{1}{2}\left\{\left(\frac{10}{2 \times 1} + 1\right)(6.66 + 5.45) - \frac{10^2}{2 \times 1}\right\} = 11.33 \text{ m}^2$$

Second section:

$$b = 10,\ h = 2.00,\ n = 5,\ s = 1$$

$$b_1 = \frac{10}{2} + \frac{5 \times 1}{5 - 1} \times \left(2 + \frac{10}{2 \times 5}\right) = 8.75 \text{ m}$$

$$b_2 = \frac{10}{2} + \frac{5 \times 1}{5 + 1}\left(2 - \frac{10}{2 \times 5}\right) = 5.83 \text{ m}$$

$$\text{Area } A_2 = \frac{1}{2}\left\{\left(\frac{10}{2 \times 1} + 2\right)(8.75 + 5.83) - \frac{10^2}{2 \times 1}\right\} = 26.03 \text{ m}^2$$

Third section:

$$b = 10 \text{ m},\ h = 1.50,\ n = 8,\ s = 1$$

$$b_1 = \frac{10}{2} + \frac{8 \times 1}{8 - 1} \times \left(1.5 + \frac{10}{2 \times 8}\right) = 7.42 \text{ m}$$

$$b_2 = \frac{10}{2} + \frac{8 \times 1}{8 + 1} \times \left(1.5 - \frac{10}{2 \times 8}\right) = 5.77 \text{ m}$$

$$\text{Area } A_3 = \frac{1}{2}\left\{\left(\frac{10}{2 \times 1} + 1.5\right)(7.42 + 5.77) - \frac{10^2}{2 \times 1}\right\} = 17.87 \text{ m}^2$$

Volume by prismoidal formula,

$$V = \frac{50}{3}\{11.33 + 17.87 + 4 \times (26.03)\} = 2{,}222.0 \text{ m}^3$$

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

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

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

Solution

At station 1:

$$h = 1.00 \text{ m},\ h_1 = 2.55 \text{ m},\ h_2 = 0.95 \text{ m},\ b = 9 \text{ m},\ b_1 = 7.50 \text{ m},\ b_2 = 5.25 \text{ m}$$

From Eq. (7),

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

$$= \left\{\frac{1.00}{2}(7.50 + 5.25) + \frac{9}{4}(2.55 + 0.95)\right\} = 14.26 \text{ m}^2$$

At station 2:

$$h = 1.5 \text{ m},\ h_1 = 2.80 \text{ m},\ h_2 = 1.35 \text{ m},\ b = 9 \text{ m},\ b_1 = 8.10 \text{ m},\ b_2 = 4.75 \text{ m}$$

$$\text{Area} = \left\{\frac{1.50}{2}(8.10 + 4.75) + \frac{9}{4}(2.80 + 1.35)\right\} = 19.01 \text{ m}^2$$

Volume according to average end area rule,

$$V = \frac{50}{2} \times \{(14.26 + 10.01)\} = 831.75 \text{ m}^3$$

Prismoidal correction,

$$C_p = \frac{D}{12}(h_1 - h_2) \text{ (whole width of one section} - \text{whole width of another section).}$$

Here
$D = 50$ m
h_1 = central height of 1st section = 1.00 m
h_2 = central height of 2nd section = 1.50 m

Width of 1st section = 7.50 + 5.25 = 12.75 m
Width of 2nd section = 8.10 + 4.75 = 12.85 m

$$C_p = \frac{50}{12}(1.00 - 1.50) \times (12.75 - 12.85) = 0.20 \text{ m}^3$$

Correct volume = 831.75 − 0.20 = 831.55 m^3

Problem 9 The formation width of a road is 10 m, and the side slope for cutting is 1 : 1 and for filling 2 : 1. The transverse slope of the ground is 1 in 5 (fall). The sections are 50 m apart. The depths of excavation at the centres of the two sections are 0.50 and 0.70 m, respectively. Find the volume of cutting and filling.

Solution Given data:

b = 10 m
n = 5
s = 1 (in cutting)
s_1 = 2 (in filling)
D = 50 m
h_1 = 0.50 m (central height at 1st section)
h_2 = 0.70 m (central height at 2nd section)

From Eq. (14),

$$\text{Area in cutting} = \frac{1}{2}\left[\frac{\{(b/2) + nh\}^2}{n - s}\right]$$

From Eq. (17),

$$\text{Area in filling} = \frac{1}{2}\left[\frac{\{(b/2) - nh\}^2}{n - s_1}\right]$$

Section 1

$$\text{Area in cutting} = \frac{1}{2}\left[\frac{\{(10/2) + 5 \times 0.5\}^2}{5 - 1}\right] = 7.03 \text{ m}^2$$

$$\text{Area in filling} = \frac{1}{2}\left[\frac{\{(10/2) - 5 \times 0.5\}^2}{5 - 2}\right] = 1.04 \text{ m}^2$$

Section 2

$$\text{Area in cutting} = \frac{1}{2}\left[\frac{\{(10/2) + 5 \times 0.7\}^2}{5 - 1}\right] = 9.03 \text{ m}^2$$

$$\text{Area in filling} = \frac{1}{2}\left[\frac{\{(10/2) - 5 \times 0.7\}^2}{5 - 2}\right] = 0.38 \text{ m}^2$$

Volume of cutting from average end area rule,

$$V = \frac{7.03 + 9.03}{2} \times 50 = 401.5 \text{ m}^3$$

Volume of filling from average end area rule,

$$V = \frac{1.04 + 0.38}{2} \times 50 = 35.5 \text{ m}^3$$

$$\text{Prismoidal correction (for cutting)} = \frac{D}{12(n - s)} \times n^2 (h_1 - h_2)^2$$

$$= \frac{50}{12(5 - 1)} \times (5)^2 (0.50 - 0.70)^2 = 1.04 \text{ m}^3$$

$$\text{Prismoidal correction (for filling)} = \frac{D}{12(n - s_1)} \times h^2 (h_1 - h_2)^2$$

$$= \frac{50}{12(5 - 2)} \times 5^2 (0.70)^2 = 1.40 \text{ m}^3$$

$$\text{Corrected volume (in cutting)} = 401.50 - 1.04 = 400.46 \text{ m}^3$$
$$\text{Corrected volume (in filling)} = 35.50 - 1.40 = 34.10 \text{ m}^3$$

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

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

Calculate the volume of earth work.

Solution

Section at station 1

Starting from the centre and running outwards to the right and left, the coordinates of the vertices are arranged irrespective of sign in determinant form, as follows:

A	B	C	D	E	F	G
$\frac{0}{3.00}$	$\frac{2.20}{5.50}$	$\frac{1.75}{3.00}$	$\frac{1.50}{0}$	$\frac{4.75}{5.25}$	$\frac{6.40}{7.30}$	$\frac{0}{3.00}$

Sum of the products of coordinates joined by solid lines,

$$\Sigma P = 4.75 \times 0 + 6.40 \times 5.25 + 0 \times 7.30 + 1.75 \times 0$$
$$+ 2.20 \times 3.00 + 0 \times 5.50$$
$$= 0 + 33.60 + 0 + 0 + 6.60 + 0 = 40.20$$

Sum of the products of coordinates joined by dotted line,

$$\Sigma Q = 1.50 \times 5.25 + 4.75 \times 7.30 + 6.40 \times 3.00 + 1.50 \times 3.00$$
$$+ 1.75 \times 5.50 + 2.20 \times 3.00$$
$$= 7.88 + 34.88 + 19.20 + 4.50 + 9.63 + 6.60 = 82.49$$

$$\text{Area} = \frac{1}{2}(\Sigma P \sim \Sigma Q) = \frac{1}{2}(40.20 - 82.49) = 21.14 \text{ m}^2$$

Section at station 2

The coordinates of the vertices are arranged in determinant form as in the case of station 1,

A_1	B_1	C_1	D_1	E_1	F_1	G_1
$\frac{0}{3.00}$	$\frac{3.10}{5.25}$	$\frac{2.20}{3.00}$	$\frac{2.00}{0}$	$\frac{5.25}{6.00}$	$\frac{7.40}{8.50}$	$\frac{0}{3.00}$

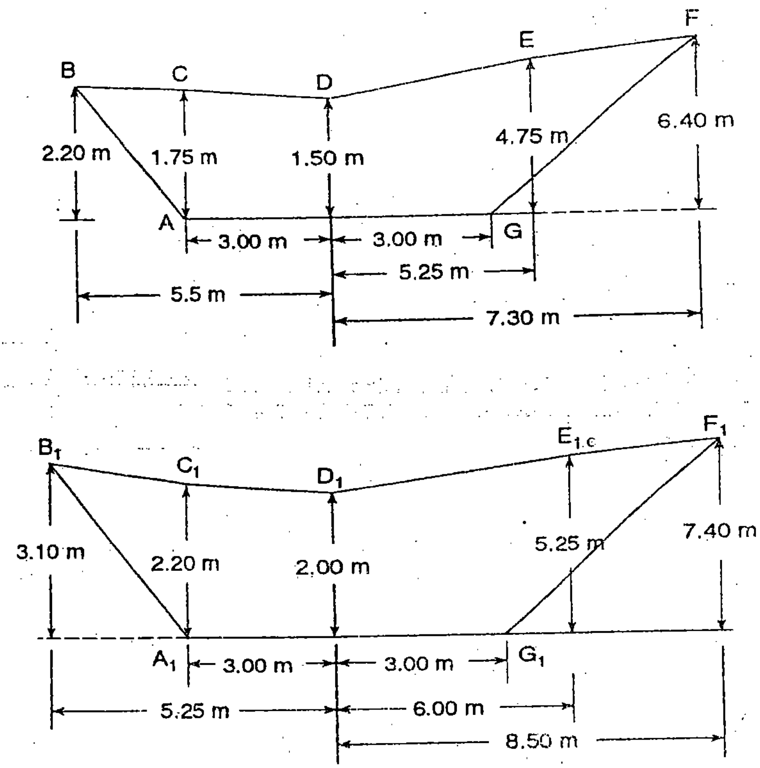


Fig. P.8.3 (a) and (b)

Sum of the products of coordinates joined by solid lines,

$$\Sigma P = 5.25 \times 0 + 7.40 \times 6.00 + 0 \times 8.50 + 2.20 \times 0 + 3.10 \times 3.00 + 0 \times 5.25$$
$$= 0 + 44.40 + 0 + 0 + 9.30 + 0 = 53.70$$

Sum of the products of coordinates joined by dotted lines,

$$\Sigma Q = 2.00 \times 6.00 + 5.25 \times 8.50 + 7.40 \times 3.00 + 2.00 \times 3.00 + 2.20 \times 5.25 + 3.10 \times 3.00$$
$$= 12.00 + 44.63 + 22.20 + 6.00 + 11.55 + 9.30 = 105.68$$

$$\text{Area} = \frac{1}{2}(\Sigma P - \Sigma Q)$$
$$= \frac{1}{2}(53.70 - 105.68) = 25.99 \text{ m}^2$$

Volume by average end area rule,

$$V = \frac{21.14 + 25.99}{2} \times 50 = 1{,}178.25 \text{ m}^3$$

SHORT QUESTIONS WITH ANSWERS FOR VIVA

Q. 1 State the different types of cross-section.

Ans. Cross-sections may be of the following types:
- (a) Level
- (b) Two-level
- (c) Three-level
- (d) Side hill two-level
- (e) Multilevel

Q. 2 Name and state the formulae for computation of volume.

Ans. There are two formulae:

(a) Trapezoidal (or average end area rule) formula—

$$\text{Volume} = \frac{\text{common distance}}{2}\{\text{area of 1st section} + \text{area of last section} + 2\,(\text{sum of the areas of other sections})\}$$

(b) Prismoidal formula—

$$\text{Volume} = \frac{\text{common distance}}{3}\{\text{area of 1st section} + \text{area of last section} + 4\,(\text{sum of areas of even sections}) + 2\,(\text{sum of areas of remaining odd sections})\}$$

Q. 3 What is the limitation of the prismoidal formula?

Ans. The prismoidal formula can be directly applied only when the number of sections is odd.

Q. 4 When would you apply prismoidal correction?

Ans. Prismoidal correction is applied when the volume is calculated by the trapezoidal formula. This correction is always subtracted from the value obtained by the trapezoidal formula.

Q. 5 What do the terms 'lead' and 'lift' mean?

Ans. The horizontal distance through which the excavated earth from the borrowpit or from cutting is transported to a nearby place is known as lead. This is the distance on which the earth work is estimated.

The vertical distance through which the excavated earth from the borrowpit or from cutting is lifted or transported is known as lift. This is also a factor to be considered while estimating earth work.

Q. 6 What is a mass diagram?

Ans. The mass diagram is a curve plotted on a base line (which represents chainages of alignment) taking the ordinates at any point as the algebraic sum of the volume of cutting and filling from the starting point up to that point. The volume of cutting is considered to be positive and that of filling negative.

The mass diagram is prepared in order to facilitate proper distribution of excavated material and selection of proper locations for borrowpits.

EXERCISES

1. A railway embankment of length 500 m, width at formation level 9 m, and side slopes 2 : 1 is to be constructed. The ground levels every 100 m along the centre line are:

Distance (m)	0	100	200	300	400	500
Ground level (m)	107.8	106.3	110.5	111.0	110.7	112.2

The embankment has a rising gradient of 1.2 m per 100 m, and the formation level is 110.5 m at zero chainage. Assuming the ground to be level across the centre line, compute the volume of earth work. (AMIE, Summer 1989)

(Ans. Trapezoidal—32,816 m^3, Prismoidal—34,803.33 m^3)

2. The areas enclosed by contour lines, at 5 m intervals, for a reservoir up to the face of a proposed dam, are shown below:

Value of contour (m)	1,005	1,010	1,015	1,020	1,025	1,030	1,035
Area (m^2)	400	1,500	3,000	8,000	18,000	25,000	40,000

Taking 1005 and 1035 m as the bottom most and highest water levels respectively, determine the capacity of the reservoir by using:
(i) the trapezoidal formula, and (ii) the prismoidal formula.

(AMIE, Winter 1986)

[Ans. (i) 378,500 m^3, (ii) 367,333.33 m^3]

3. An excavation is to be made for a reservoir 26 m long and 15 m wide at the bottom, of side slope 2 : 1. Calculate the volume of excavation if the depth is 4 m. Assume that the ground surface is level before excavation.

(AMIE, Winter 1987)

Ans. 3213.335 m^3)

4. The formation level of a road is at a constant RL of 150.00 m. The ground levels along the centre line of the road are as follows:

Chainage (m)	0	40	80	120	160	200	240
Ground level (m)	152.60	151.90	149.00	150.90	151.50	152.45	151.20

Compute the volume of earth work given that the formation width is 8 m and the side slope 2 : 1.

5. (a) Derive an expression, with the help of a neat sketch, for the area of a three-level section.
(b) A railway embankment 600 m long has a formation level width of 11.5 m with side slope 2 : 1. If the ground and formation levels are as follows, calculate the volume of earth work. The ground is level across the centre line.

Distance (m)	0	100	200	300	400	500	600
GL (m)	105.2	106.8	107.0	103.4	105.6	104.7	105.1
FL (m)	107.5	108.6	108.5	104.5	106.9	105.6	106.3

(AMIE, Summer 1988)

(Ans. 11,691.67 m^3—prismoidal, 12,091.51 m^3—trapezoidal)

6. To compute the volume of earth work between two stations 60 m apart, the following data were gathered for the three-level cross-sections at the stations:

Station	Cross-section		
A	$\frac{+1.7}{7.7}$	$\frac{+2.6}{0}$	$\frac{+4.5}{10.8}$
B	$\frac{+2.6}{8.5}$	$\frac{+3.5}{0}$	$\frac{+6.5}{12.3}$

If the width of cutting at formation level is 10.5 m, calculate the volume of cutting between the stations. (AMIE, Winter 1988)

(Ans. 3,008.25 m^3)

7. The formation width of a road is 8 m and the side slope is 2 : 1. The surface of the ground has a transverse slope of 1 in 10. If the depths of cutting at the centres of three sections 40 m apart are 1.5, 2.5 and 2.0 m respectively, determine the volume of earth work. (Ans. 1,808.27 m^3)

8. The width of a hilly road at formation level is 8 m, and the side slope is 1 : 1 in cutting and 2 : 1 in filling. The ground surface has a transverse slope of 1 in 5. If the distance between two sections is 50 m and depths of excavation at the centre are 0.75 and 1.25 respectively, find the volume of cutting and filling.

(Ans. Cutting—509.49 m^3, filling—12.57 m^3)

9. The following are the data pertaining to an irregular cross-section of two stations 50 m apart. The width of the road is 8 m and the side slope is 1 : 1. Calculate the volume of earth work.

Station	Left		Centre	Right	
1	$\frac{+3.25}{5.35}$	$\frac{+2.20}{4.00}$	$\frac{+3.75}{0}$	$\frac{+6.00}{5.8}$	$\frac{+7.2}{8.9}$
2	$\frac{+4.35}{5.75}$	$\frac{+2.50}{4.00}$	$\frac{+2.90}{0}$	$\frac{+4.75}{6.0}$	$\frac{+5.50}{9.20}$

(Ans. 2,063 m^3)

10. Choose the correct alternative in questions (i) to (viii).
(i) The volume computed by the prismoidal method is considered to be
(a) Exact (b) Approximate (c) Average
(ii) To obtain the correct volume using the trapezoidal rule, the prismoidal correction should always be
(a) Added (b) Subtracted (c) Multiplied
(iii) The horizontal distance through the excavated earth transported from the borrowpit to the embankment is known as
(a) Lift (b) Hand distance (c) Lead
(iv) The vertical distance through which excavated earth is lifted is called
(a) Lead (b) Lift (c) Haulage
(v) The graph prepared in order to facilitate proper distribution of excavated earth is known as the
(a) Mass diagram (b) Working diagram (c) Balancing diagram
(vi) With notations carrying their usual meanings, the cross-sectional area of an embankment is given by
(a) $(b + 2sh)h$ (b) $(b + sh)h$ (c) $(b + sh)h^2$
(vii) With notations carrying their usual meanings, the cross-sectional area in a two-level section is given by

(a) $\left\{\left(\frac{b}{25} + h\right) - \frac{b^2}{25}\right\}^2$ (b) $\frac{1}{2}\left\{\left(\frac{b}{25} + h\right)(b_1 + b_2) - \frac{b^2}{25}\right\}$

(c) $\left\{\left(\frac{b}{25} + h\right) - \frac{b^2}{25}\right\}$

(viii) With notations carrying their usual meanings, the prismoidal correction for a level section is given by

(a) $\frac{D \times s}{6}(h_1 - h_2)^2$ (b) $\frac{D \times s}{6}(h_1^2 - h_2^2)$ (c) $\frac{D \times s}{3}(h_1 + h_2)$

ANSWERS

Q. 10	(i) a	(ii) b	(iii) c	(iv) b
	(v) a	(vi) b	(vii) b	(viii) a

9

Theodolite Traversing

9.1 INTRODUCTION

The theodolite is an intricate instrument used mainly for accurate measurement of horizontal and vertical angles up to 10″ or 20″, depending upon the least count of the instrument. Because of its various uses, the theodolite is sometimes known as a universal instrument. The following are the different purposes for which the theodolite can be used:

1. Measuring horizontal angles
2. Measuring vertical angles
3. Measuring deflection angles
4. Measuring magnetic bearings
5. Measuring the horizontal distance between two points
6. Finding the vertical height of an object
7. Finding the difference of elevation between various points
8. Ranging a line

Theodolites may be of two types—(i) transit theodolite, and (ii) non-transit.

In the transit theodolite, the telescope can be revolved through a complete revolution about its horizontal axis in a vertical plane.

In the non-transit theodolite, the telescope cannot be revolved through a complete revolution in the vertical plane. But it can be revolved to a certain extent in the vertical plane, in order to measure the angle of elevation or depression.

Theodolites may also be classified as: (i) vernier theodolites—when fitted with a vernier scale, and (ii) micrometer theodolites—when fitted with a micrometer.

The size of the theodolite is defined according to the diameter of the main horizontal graduated circle. For example, in a "10 cm theodolite", the diameter of the main graduated circle is 10 cm. In engineering survey, 8 cm to 12 cm theodolites are generally used.

9.2 DEFINITIONS

1. Centring The setting of a theodolite exactly over a station mark by means of a plumb-bob is known as centring. The plumb-bob is suspended from a hook fixed below the vertical axis.

2. Transiting The method of turning the telescope about its horizontal axis in a vertical plane through 180° is termed as transiting. In other words, transiting results in a change in face.

3. Face left 'Face left' means that the vertical circle of the theodolite is on the left of the observer at the time of taking readings.

The observation taken in the face left position is called faced left observation.

4. Face right This refers to the situation when the vertical circle of the instrument is on the right of the observer when the reading is taken. The observation taken in the face right position is known as face right observation.

5. Telescope normal The face left position is known as 'telescope normal' or 'telescope direct'. It is also referred to as 'bubble up'.

6. Telescope inverted The face right position is called 'telescope inverted' or 'telescope reversed'. It is also termed 'bubble down'.

7. Changing face The operation of bringing the vertical circle from one side of the observer to the other is known as changing face.

8. Swinging the telescope This indicates turning of the telescope in a horizontal plane. It is called 'right swing' when the telescope is turned clockwise and 'left swing' when the telescope is turned anticlockwise.

9. Line of Collimation It is an imaginary line passing through the intersection of the cross hairs at the diaphragm and the optical centre of the object glass and its continuation.

10. Axis of the telescope This axis is an imaginary line passing through the optical centre of the object glass and the optical centre of the eye-piece.

11. Axis of the bubble tube It is an imaginary line tangent.al to the longitudinal curve of the bubble tube at its middle point.

12. Vertical axis It is the axis of rotation of the telescope in the horizontal plane.

13. Horizontal axis It is the axis of rotation of the telescope in the vertical plane. It is also known as the turnnion axis.

14. Temporary adjustment The setting of the theodolite over a station at the time of taking any observation is called temporary adjustment. This adjustment is necessary for every set up of the instrument. (See Sec. 9.5 for a detailed description.)

15. Permanent adjustment When the desired relationship between the fundamental lines of a theodolite is disturbed, then some procedures are adopted to establish this relationship. This adjustment is known as permanent adjustment. (This is described in detail in Sec. 9.21.)

16. Least count of the vernier This is the difference between the value of the smallest division of the main scale and that of the smallest division of the vernier scale. It is the smallest value that can be measured by a theodolite.

Let $(n - 1)$ small divisions of the main scale be divided into n small divisions in the vernier scale.

Then $$n \times v = (n - 1)d$$

$$\therefore \quad v = \frac{(n-1)\,d}{n}$$

where v = Value of smallest division of vernier scale
d = Value of smallest division of main scale

So, according to the definition

$$\text{Least count} = d - v = \frac{d - (n-1)d}{n} = \frac{nd - nd + d}{n} = \frac{d}{n}$$

Illustration

If $$d = 20' \text{ and } n = 60;$$

$$\text{Least count} = \frac{20}{60} \times 60 = 20''$$

If $$d = 15' \text{ and } n = 60,$$

$$\text{Least count} = \frac{15}{60} \times 60 = 15''$$

and so on,

17. Magnification or magnifying power of telescope The magnifying power of a telescope is the ratio of the focal length of the objective to that of the eye-piece.

If f = focal length of objective
f_1 = focal length of eye-piece

$$\text{Magnifying power} = \frac{f}{f_1}$$

It is expressed in terms of the diameter, e.g. 20 diameters, 30 diameters, etc.

18. The diaphragm The diaphragm is a brass ring consisting of cross-hairs, or one containing a glass disc with fine lines engraved on it. It is placed in position by turning four capstan-headed screws, and can be moved up, down or sideways when required. It is fixed in front of the eye-piece. The cross-hairs may be made of spider web or fine platinum wire; they may also be in the form of a fine scratch mark engraved on glass.

Figure 9.1 shows the different types of arrangement of cross-hairs.

The vertical hair or hairs are meant for checking the verticality of the staff or target. The middle horizontal hair corresponds to the collimation line. The upper and lower short horizontal lines are called the upper and lower stadia.

19. Sensitiveness of bubble tube The ability of a bubble tube to show a very

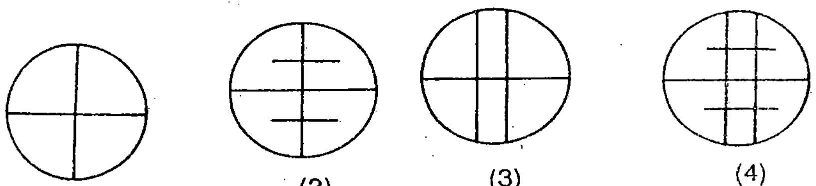


Fig. 9.1

small deviation of the bubble from its horizontal position is termed as the sensitiveness of the bubble tube. It depends on the following factors:

(a) The radius of curvature of the internal surface of the tube.
(b) The diameter of the bubble tube.
(c) The length of the bubble tube.
(d) The viscosity of the liquid used inside the tube.

Sensitiveness is expressed in terms of the angle by which the axis of the bubble will be tilted for a deviation of the bubble by one small division.

$$\text{Angular value } \alpha' = \frac{s}{Dn} \text{ radians}$$

$$= \frac{s}{Dn} \times 206{,}265'' \qquad (1 \text{ radians} = 206{,}265'')$$

where s = Staff intercept
D = Distance between instrument and staff
n = Number of divisions through which the bubble moves
(Sensitiveness is described in detail in Sec. 5.8.)

9.3 THE TRANSIT THEODOLITE

The following are the essential parts of a theodolite:

1. Trivet It is a circular plate having a central, threaded hole for fixing the theodolite on the tripod stand by a wing nut. It is also called the base plate. Three foot screws are secured to this plate by means of a ball-and-socket arrangement.

2. Foot screws These are meant for levelling the instrument. The lower part of the foot screws are secured in the trivet by means of a ball-and-socket arrangement, and the upper threaded part passes through the threaded hole in the tribrach plate.

3. Tribrach It is a triangular plate carrying three foot screws at its ends.

4. Levelling head The trivet, foot screws and the tribrach constitute a body which is known as the levelling head.

5. Spindles The theodolite consists of two spindles or axes—one inner and the other outer. The inner axis is solid and conical, and the outer is hollow. The two spindles are coaxial.

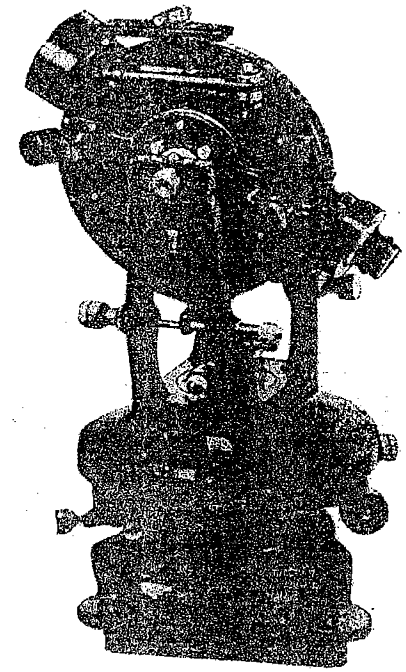


Photo Transit Vernier Theodolite 20″

6. Lower plate The lower plate is attached to the outer axis, and is also known as the scale plate. It is bevelled and the scale is graduated from 0 to 360° in a clockwise direction. Each degree is again subdivided into two, three or four divisions; thus, the value of one small division may be 30′, 20′ or 15′ respectively.

The lower plate is provided with a clamp screw and a tangent screw which control its movements. When the clamp screw is tightened, this plate is fixed with the outer axis. For fine adjustment of the lower plate, the tangent screw is rotated to the extent required. The size of the theodolite is designated according to the diameter of the lower plate.

7. Upper plate The upper plate contains the vernier scales A and B. It is attached to the inner axis. Its motion is controlled by the upper clamp screw and the upper tangent screw. When the clamp screw is tightened the vernier scales are fixed with the inner axis, and for fine adjustment of the scales the tangent screw is rotated.

8. Plate bubble Two plate bubbles are mounted at right angles to each other on the upper surface of the vernier plate. One bubble is kept parallel to the horizontal axis of the theodolite. Sometimes one plate bubble is provided on the vernier plate. The bubbles are meant for levelling the instrument at the time of measuring the horizontal angles.

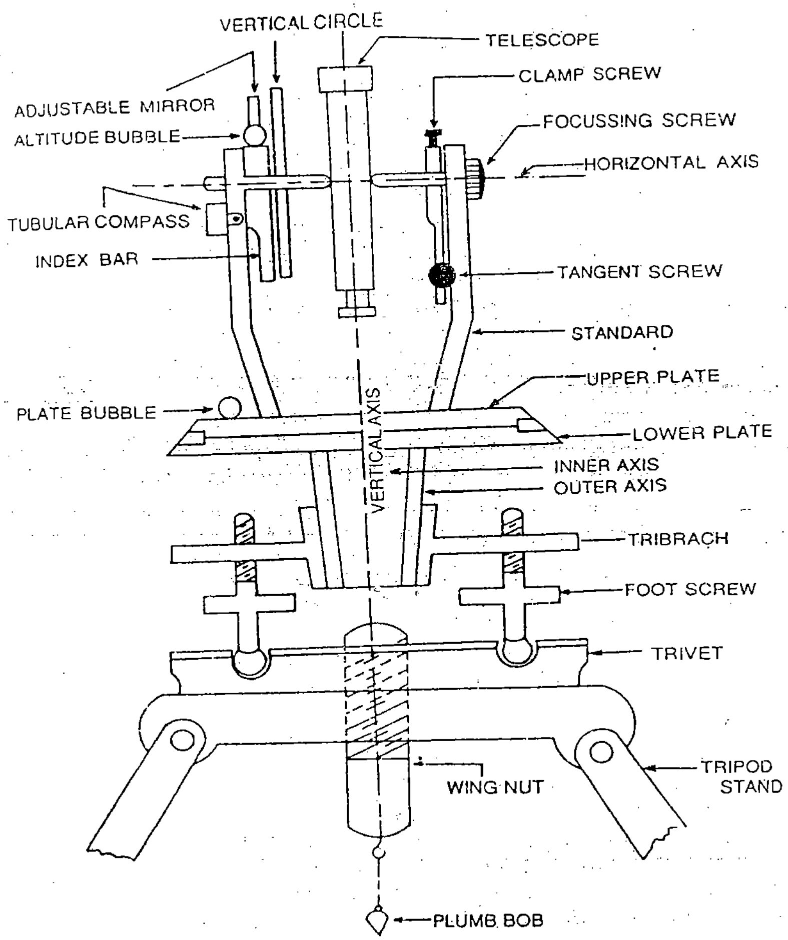


Fig. 9.2

9. Standard or 'A' frame Two frames (shaped like the letter 'A') are provided on the upper plate to support the telescope, the vertical circle and the vernier scales. These frames are known as standards or A-frames.

10. The telescope The telescope is pivoted between the standards at right angles to the horizontal axis. It can be rotated about its horizontal axis in a vertical plane. The telescope is provided with a focussing screw, clamping screw and tangent screw.

11. Vertical circle The vertical circle is rigidly fixed with the telescope and moves with it. It is divided into four quadrants. Each quadrant is graduated from 0 to 90° in opposite directions, with the 'zero' mark at the ends of the horizontal diameter of the vertical circle.

The line joining the 'zero' marks corresponds to the line of collimation. The subdivisions of the vertical circle are similar to these of the horizontal circle. The vertical circle can be clamped or finely adjusted with the help of the clamping screw and the tangent screw provided along with the telescope.

12. Index bar or T frame The index bar is provided on the standard in front of the vertical circle. It carries two verniers (C and D) at the two ends of the horizontal arm. The vertical leg of the index bar is provided with a clip screw at the lower end by means of which the altitude bubble can be brought to the centre.

13. Altitude bubble A long sensitive bubble tube is provided on the top of index bar. The bubble it contains is known as the altitude bubble. This bubble is brought to the centre by the clip screw at the time of measuring the vertical angle. A mirror is provided on the top of the bubble to help observe it when the instrument is set up above normal height.

14. Compass Sometimes a circular box compass is mounted on the vernier scale between the standards. In modern theodolites, an adjustable trough compass or tubular compass can be fitted with a screw to the standard. The compass is provided for taking the magnetic bearing of a line.

9.4 READING THE VERNIER THEODOLITE

The least count of the vernier is to be determined first. Let it be 20″. The main division of the main scale is of one degree. Suppose it is divided into three parts. Then each part accounts for 20′ (i.e. $d = 20'$).

The vernier scale has 20 big and 60 small divisions.

So, $$\text{Least count} = \frac{d}{n} = \frac{20}{60} \times 60 = 20''$$

Here, Least count for one small division = 20″

∴ Least count for one big division = (20″ × 3) = 60″ = 1′.

After making the final adjustment for measuring the angle, the position of the arrow of the vernier scale is noted. Suppose the arrow crosses 10° and 20′, which

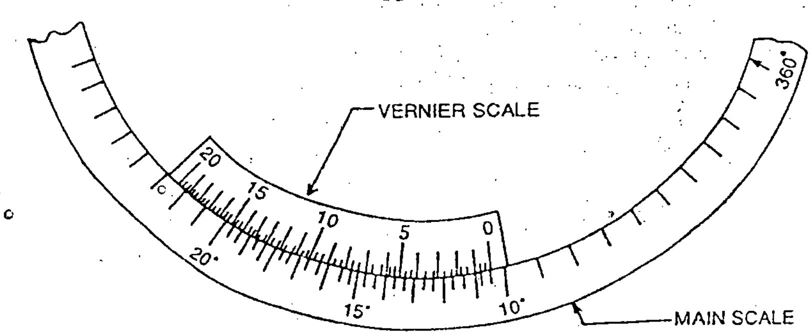


Fig. 9.3

is the the direct reading obtained from the main scale. Suppose, again, that the first small division after 12 big divisions exactly coincides with any of the main scale divisions. Then, the vernier reading 12′20″.

$$\therefore \quad \text{Final angle} = 10°20' + 12'20'' = 10°32'20'$$

9.5 READING THE MICROMETER THEODOLITE

The micrometer theodolite gives an accurate result more quickly than the vernier scale.

Each main division of the main scale corresponds to one degree, and is subdivided into six parts. So, each subdivision represents 10′. The drum of the micrometer is divided into ten main divisions, each corresponding to one minute. Further, each main division of the drum is subdivided into six parts. So, each part corresponds to 10″.

The micrometer consists of a diaphragm which carries two parallel vertical hair lines. The hair lines are moved by a screw. It also consists of an index mark (V-notch). At the beginning, the micrometer is set to zero, when the index mark is just within the hair lines.

For measuring any angle, the position of the index mark on the main scale is noted. Suppose it crosses 31°30′, which is the reading directly obtained from the main scale [Fig. 9.4(a)].

Now, the micrometer is moved backwards until the next backward division (i.e. the one corresponding to 30′) is exactly midway between the hairs. At this time, the micrometer reading is taken. Let the reading be 5′10″ [Fig. 9.4(b)].

$$\text{So,} \quad \text{Final angle} = 31°30' + 5'10'' = 31°35'10''$$

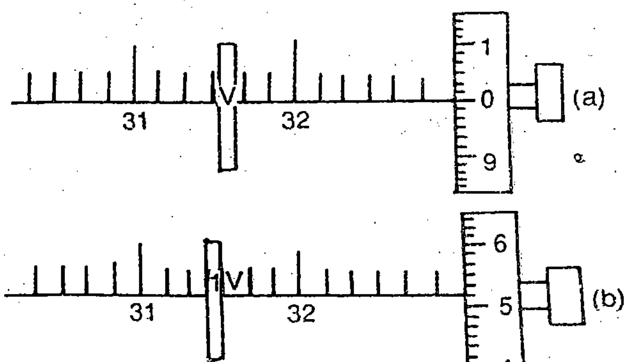


Fig. 9.4 (a), (b)

9.6 TEMPORARY ADJUSTMENT OF THEODOLITE

Such adjustment involves the following steps.

1. Setting the theodolite over the station The tripod stand is placed over the required station. The theodolite is then lifted from the box and fixed on top of the stand by means of a wing nut or according to the fixing arrangement provided along with the instrument.

2. Approximate levelling by tripod stand The legs of the tripod stand are placed well apart and firmly fixed on the ground. Then, approximate levelling is done using this stand. To do this, two legs are kept firmly fixed on the ground and the third is moved in or out, clockwise or anticlockwise, so that the bubble is approximately at the centre of its run.

3. Centring Centring is the process of setting the instrument exactly over a station. At the time of approximate levelling by means of the tripod stand, it should be ensured that the plumb bob suspended from the hook under the vertical axis lies approximately over the station peg.

Then, with the help of the shifting head (movable capstan nut), the centring is done accurately so that the plumb bob is exactly over the nail of the station peg.

4. Levelling Before starting the levelling operation, all the foot screws are brought to the centre of their run. Then the following procedure is adopted:

(a) The plate bubble is placed parallel to any pair of foot screws (say the first and second foot screws). By turning both these screws equally inwards or outwards, the bubble is brought to the centre.

(b) The plate bubble is turned through 90° so that it is perpendicular to the line joining the first and second foot screws. Then by turning the third foot screw either clockwise or anticlockwise the bubble is brought to the centre (Fig. 9.5).

Some instruments may have two plate bubbles perpendicular to each other. In such a case, one bubble is kept parallel to any pair of foot screws; the other plate bubble will automatically be perpendicular to the position of the first bubble. Here, the instrument need not be turned. The first bubble can be brought to the centre by turning the first and second foot screws, and the second bubble can be brought to the centre by turning the third foot screw.

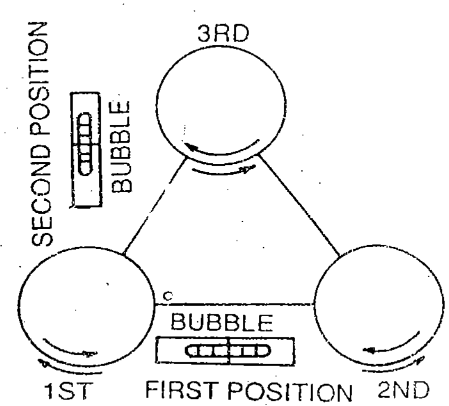


Fig. 9.5

(c) The process is repeated several times, so that the bubble remains in the central position of the plate bubble, both directions perpendicular to each other.

(d) The instrument is rotated through 360° about its vertical axis. If the bubble still remains in the central position, the adjustment of the bubble is perfect and the vertical axis is truly vertical.

5. Focussing the eye-piece The eye-piece is focussed so that the cross-hairs can be seen clearly. To do this, the telescope is directed towards the sky or a piece of white paper is held in front of the object glass, and the eye-piece is moved in or out by turning it clockwise or anticlockwise until the cross-hairs appear distinct and sharp.

6. Focussing the object glass This is done to bring a sharp image of the object or target in the plane of cross-hairs and to eliminate parallax. To do this, the telescope is directed towards the object or target and the focussing screw is turned clockwise or anticlockwise until the image appears clear and sharp and there is no relative movement between the image and cross-hairs. The absence of relative movement can be verified by moving the eye up and down.

7. Setting the vernier The vernier A is set to 0° and vernier B to 180°. To do this, the lower clamp is fixed. The upper clamp is loosened and the upper plate turned until the arrow of vernier A approximately coincides with zero (i.e. the 360° mark) and that of vernier B approximately coincides with the 180° mark. Then the upper clamp is tightened, and by turning the upper tangent screw the arrows are brought to a position of exact coincidence.

This completes the temporary adjustment required for taking readings with the theodolite.

Note: Instruments with four foot screws are absolete now-a-days. However, the procedure of levelling involves making the bubble parallel to the line joining the foot screws AB and CD successively, and by turning these foot screws equally inwards or outwards the bubble is brought to the centre of its run.

The process is repeated several times and the permanent adjustment of the instrument is tested by turning the bubble through 180°.

9.7 SOME MODERN THEODOLITES

Geodetic and astronomical surveys require a high degree of precision. In order to meet this need, high-precision theodolites are manufactured now-a-days. The characteristic features of modern theodolites are as follows:

1. They are more compact and light.
2. The graduations are made on a glass circle, and are finer.
3. Improved micrometers, using which the observer can take readings accurate to one second, are provided along with them.
4. The instrument is made water-proof and dustproof.
5. It is electrically illuminated to facilitate work at night or in tunnels.
6. Adjustments for the micrometer are not necessary.
7. Magnification is higher by about 40 diameters.

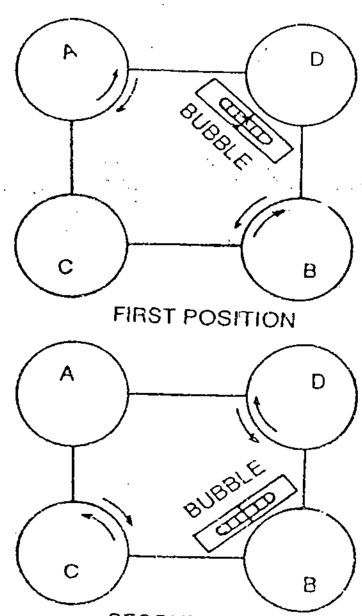


Fig. 9.6

We shall now describe a few high-precision theodolites.

1. Watts micro-optic theodolite There are three models of this type. The first and third models are capable of reading up to 5″, and the second can read up to 1″.

The horizontal and vertical circles of this theodolite are made of glass. Micrometers for measuring horizontal and vertical angles are provided. The other accessories are the same as in the transit theodolite. But the arrangements are very compact, and well protected from atmospheric action.

This instrument is manufactured by M/s Hilger and Watt Ltd. The procedure for operation is described in detail in the catalogue supplied by the manufacturer.

2. Wild T-2 theodolite The horizontal and vertical circles of this instrument are made of glass. The diameter of the horizontal circle is 90 mm and that of the vertical circle 70 mm. The circles are electrically illuminated through an adjustable mirror. The length of the telescope is 148 mm. The vertical axis consists of an axle bush having ball bearings. The instrument is automatically centred by its own weight. The readings are taken through a micrometer by the coincidence system.

The theodolite is manufactured by M/s Wild Heerbrugg Ltd. Detail of operation are available in the catalogue.

3. Wild T-3 precision theodolite The horizontal and vertical circles are made of glass and are finely graduated. The minimum reading of the horizontal circle is 4′ and that of the vertical circle 8′. The angle is measured by means of an optical micrometer, which is accurate up to 0.2″. The vertical axis consists of an axle bush and ball bearings. The instrument is automatically centred by its own weight. It consists of one set of clamp and tangent screws for the motion of the vertical axis.

This instrument is manufactured by M/s Wild Heerbrugg Ltd. The procedure for operation is described in the catalogue.

4. Wild T-4 universal theodolite This instrument is widely used in the determination of geographical positions, and for taking astronomical observations with the utmost precision. It consists of a horizontal circle of diameter 250 mm and graduated to a minimum reading of 2′. With the optical micrometer, one can take readings as low as 0.1″. The vertical and horizontal circles are equipped with a "reading micrometer" which gives the arithmetic mean of two diametrically opposite readings automatically. This instrument is manufactured by M/s Wild Heerbrugg Ltd.

5. The tavistock theodolite The horizontal and vertical circles are made of glass and are so graduated that a reading as low as 1″ can be taken, and one of 0.25″ can be estimated. A single optical micrometer is provided for both the scales. Both circles are illuminated by a single mirror. It is provided with an optical plummet for centring over the station.

The Tavistock theodolite is manufactured by M/s Vickers Instrument Ltd., England.

Photo Wild T-3 Theodolite

9.8 DIRECT METHOD OF MEASURING HORIZONTAL ANGLE

Consider Fig. 9.7. Suppose an angle ∠AOB is to be measured. The following procedure is adopted:

1. The instrument is set up over O. It is centred and levelled perfectly according to the procedure described for temporary adjustment. Suppose the instrument was initially in the face left position.
2. The lower clamp is kept fixed. The upper clamp is loosened, and by turning the telescope clockwise vernier A is set to 0° and vernier B to approximately 180°.

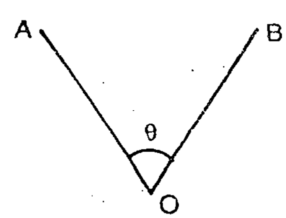


Fig. 9.7

The upper clamp is then tightened. Now by turning the upper tangent screw, verniers A and B are set to exactly 0° and 180° by looking through the magnifying glass.

3. The upper clamp is tightly fixed. The lower one is loosened and the telescope is directed to the left-hand object A. The ranging rod at A is bisected approximately by properly focussing the telescope and eliminating parallax. The lower clamp is tightened, and by turning the lower tangent screw the ranging rod at A is accurately bisected.
4. The lower clamp is kept fixed. The upper clamp is loosened and the telescope is turned clockwise to approximately bisect the ranging rod at B by properly focussing the telescope. The upper clamp is tightened, and the ranging rod at B bisected accurately by turning the upper tangent screw.
5. The readings on verniers A and B are noted. Vernier A gives the angle directly. But in the case of vernier B, the angle is obtained by subtracting the initial reading from the final reading. The readings are noted in tabular form, as in Table 9.1.
6. The face of the instrument is changed and the previous procedure is followed. The readings of the verniers are noted in the table.
7. The mean of the observations (i.e. face left and face right) is the actual angle ∠AOB. The two observations are taken to eliminate any possible error due to imperfect adjustment of the instrument.

There are two methods of measuring horizontal angles—those of repetition and reiteration.

A. Repetition Method

In this method, the angle is added a number of times. The total is divided by the number of readings to get the angle. The angle should be measured clockwise in the face left and face right positions, with three repetition at each face. The final reading of the first observation will be the initial reading of the second observation, and so on. The following procedure should be adopted:

Procedure

1. Suppose the angle ∠AOB (Fig. 9.8) is to be measured by the repetition process. The theodolite is set up at O. The instrument is centred and levelled properly. Vernier A is set to 0° and vernier B to 180°.
2. The upper clamp is fixed, and the lower one loosened. By turning the telescope, the ranging rod at A is perfectly bisected with the help of the lower clamp screw and the lower tangent screw. Here the initial reading of vernier A is 0°.
3. The upper clamp is loosened and the telescope is turned clockwise to perfectly bisect the ranging rod at B. The upper clamp is clamped. Suppose the reading on vernier A is 30°.
4. The lower clamp is loosened and the telescope turned anticlockwise to exactly bisect the ranging rod at A. Here, the initial reading is 30° for the second observation.

Table 9.1 *Measurement of horizontal angle*

Station	Object	Angle	Observation	Reading on vernier A	Reading on vernier B	Angle on vernier A	Angle on vernier B	Mean angle of vernier	Mean angle of observation	Remark
1	2	3	4	5	6	7	8	9	10	11
O	A	∠AOB	Face left	0°0′0″	180°0′0″					
	B			30°20′20″	210°20′40″	30°20′20″	30°20′40″	30°20′30″	30°20′30″	
O	A	∠AOB	Face right	0°0′0″	180°0′0″					
	B			30°20′40″	210°20′20″	30°20′40″	30°20′20″	30°20′30″		

5. The lower clamp is tightened. The upper one is loosened and the telescope is turned clockwise to exactly bisect the ranging rod at B. Let the reading on vernier A be 60°.
6. The initial reading for the third observation is set to 60°. ∠AOB is again measured. Let the final reading on the vernier A be 90°, which is the accumulated angle.

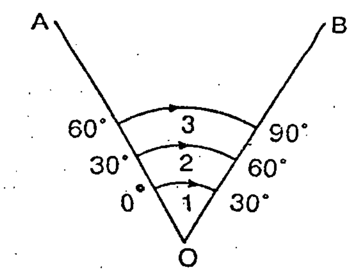


Fig. 9.8

$$\angle AOB = \frac{\text{accumulated angle}}{\text{no. of reading}} = \frac{90^\circ}{3} = 30^\circ$$

7. The face of the instrument is changed and the previous procedure is followed.
8. The mean of the two observations gives the actual angle ∠AOB. The result is shown in Table 9.2.

B. Reiteration Method

This method is suitable when several angles are measured from a single station. In this method all the angles are measured successively and finally the horizon is closed (i.e. the angle between the last station and first station is measured.) So, the final reading of the leading vernier should be the same as its initial reading. If the discrepancy is small, the error is equally distributed among all the observed angles. If it is large, the readings should be cancelled and new sets taken.

Suppose it is required to measure ∠AOB and ∠BOC from station O. The procedure is as follows.

First set

1. The theodolite is perfectly centred over O (Fig. 9.9) and levelled properly in the usual manner. Suppose, the observation is taken in the face left position and the telescope is turned clockwise (right swing).
2. Vernier A is set to 0° (i.e. 360°) and vernier B to 180°.
3. The upper clamp is fixed and the lower one loosened. The ranging rod at A is perfectly bisected. Now, the lower clamp is tightened.
4. The upper clamp is loosened, and the ranging rod or object at B is bisected properly by turning the telescope clockwise. The readings on both the verniers are taken. ∠AOB is noted.
5. Similarly, the object C is bisected properly, and the readings on the verniers are noted. ∠BOC is recorded.
6. Now the horizon is closed, i.e. the last angle ∠COA is measured. The position of the leading vernier is noted. The leading vernier should show the initial reading on which it was set. If it does not, the amount of discrepancy is noted. If it is small, the error is distributed among the angles. If the discrepancy large, the observation should be taken again.

Table 9.2 *Measurement of angle by repetition process*

Station	Object	Face	Angle	No. of readings	Initial angle on vernier A	Initial angle on vernier B	Final reading on vernier A	Final reading on vernier B	Angle on vernier A	Angle on vernier B	Mean angle of vernier	Mean angle of observation	Remark
1	2	3	4	5	6	7	8	9	10	11	12	13	14
O	A B	Left	∠AOB	1 2 3	0°0'0" 30°40'0" 61°40'0"	180°0'0" 210°40'20" 240°20'20"	30°40'0" 61°40'0" 91°20'20"	210°40'20" 240°20'20" 271°0'20"	30°26'46"	30°20'6"	30°23'26"	30°23'24"	
O	A B	Right	∠AOB	1 2 3	0°0'0" 30°40'20" 61°40'20"	180°0'0" 210°40'0" 240°20'40"	30°40'20" 61°40'20" 91°20'20"	210°40'0" 240°20'40" 271°0'0"	30°26'46"	30°20'0"	30°23'23"		

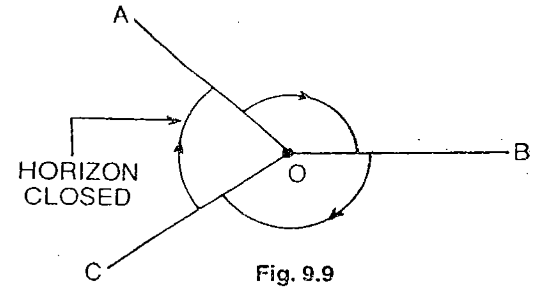


Fig. 9.9

Second set

1. The face of the instrument is changed. Again the verniers are set at their initial positions. This time the angles are measured anticlockwise (left swing).
2. The upper clamp is fixed, and the lower one loosened. Then the object A is perfectly bisected.
3. The lower clamp is tightened. The telescope is turned anticlockwise, and the object C bisected by loosening the upper clamp screw. The readings on both the verniers are taken. ∠COA is noted.
4. Then the object B is bisected by turning the telescope anticlockwise, and the readings on the verniers are taken. ∠BOC is recorded.
5. Finally, the horizon is closed i.e. the object A is bisected. Here, the leading vernier A should show a reading of 0° (since it was initially set to 0°). The last angle ∠AOB is noted.

The mean angles of the two sets give the actual values of the angles. If some error is found after arithmetical check, it should be equally distributed among the angles. The observations are noted as shown in Table 9.3.

9.9 MEASURING VERTICAL ANGLE

The vertical angle is the one between the horizontal line (i.e. line of collimation) and the inclined line of sight. When it is above the horizontal line, it is known as the angle of elevation. When this angle is below the horizontal line, it is called the angle of depression.

Consider Fig. 9.10. Suppose the angle of elevation ∠AOC and that of depression ∠BOC are to be measured. The following procedure is adopted:

1. The theodolite is set up at O. It is centred and levelled properly. The zeros of the verniers (generally C and D) are set at the 0°–0° mark of the vertical circle (which is fixed to the telescope). The telescope is then clamped.
2. The plate bubble is brought to the centre with the help of foot screws (in the usual manner). Then the altitude bubble is brought to the centre by means of a clip screw. At this position the line of collimation is exactly horizontal.
3. To measure the angle of elevation, the telescope is raised slowly to bisect the point A accurately. The readings on both the verniers are noted, and the angle of elevation recorded.

Table 9.3 (Measurement of Angle by) Reiteration Process

Inst. Station	Station bisected	Observation	Reading on vernier A	Reading on vernier B	Angle	Angle on vernier A	Angle on vernier B	Mean angle of vernier A and B	Mean angle of observation	Correction (–ve)	Corrected angle	Remark
1	2	3	4	5	6	7	8	9	10	11	12	13
O	A	Face left, right swing (clockwise)	0°0'0"	180°0'0"								
	B		100°40'40"	280°40'20"	∠AOB	100°40'40"	100°40'20"	100°40'30"	∠AOB = 100°40'15"	– 5"	∠AOB = 100°40'10"	The error is distributed equally among all the angles
	C		161°20'40"	341°20'40"	∠BOC	60°40'0"	60°40'20"	60°40'10"	∠BOC = 60°40'20"	– 5"	∠BOC = 60°40'15"	
	A		0°0'20"	180°0'20"	∠COA	198°39'40"	198°39'40"	198°39'40"	∠COA = 198°39'40"	– 5"	∠COA = 198°39'35"	
O	A	Face right, left swing (anticlock-wise)	0°0'0"	180°0'0"								
	C		161°20'20"	341°20'20"	∠COA	198°39'40"	198°39'40"	198°39'40"				
	B		100°40'20"	280°39'40"	∠BOC	60°40'20"	60°40'40"	60°40'30"				
	A		0°0'0"	180°0'0"	∠AOB	100°40'20"	100°39'40"	100°40'0"				

Total angle = 360°0'15"
Error = + 15"
Correction per angle = – 5"

Total angle = 360°0'0"

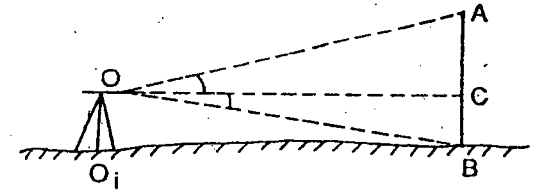


Fig. 9.10

4. The face of the instrument is changed and the point A is again bisected. The readings on the verniers are noted. The mean of the angles of the observed is assumed to be the correct angle of elevation.
5. To measure the angle of depression, the telescope is lowered slowly and the point B is bisected. The readings on the verniers are noted for the two observations (face left and face right). The mean angle of the observation is taken to be the correct angle of depression. The result is tabulated as shown in Table 9.4.

Note: The readings on the verniers should be taken carefully by noting the 'zero' position of the main scale. The verniers are graduated in both directions. Therefore, it should be ensured that the readings are taken along the proper direction.

9.10 MEASUREMENT OF DEFLECTION ANGLE

The deflection angle is the angle by which a line is deflected from its original direction. In other words, it is the angle which a survey line makes with the extension of the preceding line. The deflection may be towards the right or the left, depending upon whether the angle is measured in the clockwise or anticlockwise direction from the extension of the preceding line. Deflection angles are measured for designing horizontal curves in railways, highways, etc.

Consider Fig. 9.11. Let AB be the general direction of survey. Suppose it is deflected in the direction BC. The line AB is extended up to P. Then ∠PBC (ϕ) is known as the angle of deflection, and has to be measured. The following procedure is adopted:

1. The theodolite is set up at B, and centred and levelled properly. Vernier A is set at 0° and B at 180°.
   The upper clamp is tightened and the lower one loosened. By turning the telescope, the ranging rod at A is perfectly bisected. The lower clamp is then fixed.
2. The telescope is transited and a ranging rod at P is fixed along the prolongation of AB.
3. The upper clamp is loosened. By turning the telescope clockwise, the ranging rod at C is properly bisected. The readings on both the verniers are taken. The upper clamp is then tightened. Now, the verniers give the deflection angle ϕ.

Table 9.4 Measurement of vertical angle

Inst. Station	Object	Face	Angle	Reading on vernier C	Reading on vernier D	Angle on vernier C	Angle on vernier D	Mean angle of vernier	Mean angle of observation	Remark
1	2	3	4	5	6	7	8	9	10	11
O	Horizontal A	Left	— Elevation ∠AOC	0°0'0" 10°40'20"	0°0'0" 10°40'40"	10°40'20"	10°40'40"	10°40'30"	10°40'35"	Elevation
O	Horizontal A	Right	— Elevation ∠AOC	0°0'0" 10°40'40"	0°0'0" 10°40'40"	10°40'40"	10°40'40"	10°40'40"		
O	Horizontal B	Left	— Depression ∠BOC	0°0'0" 8°20'20"	0°0'0" 8°21'0"	8°20'20"	8°21'0"	8°20'40"	8°21'25"	Depression
O	Horizontal B	Right	— Depression ∠BOC	0°0'0" 8°22'40"	0°0'0" 8°21'40"	8°22'40"	8°21'40"	8°22'10"		

4. The lower clamp is loosened, and by turning the telescope clockwise the ranging rod at A is again bisected. (At this time the vernier readings are left undisturbed.) The lower clamp is fixed.
5. The telescope is transited. The upper clamp is loosened and by turning the telescope clockwise the ranging rod at C is bisected once more. The readings on the vernier are taken.

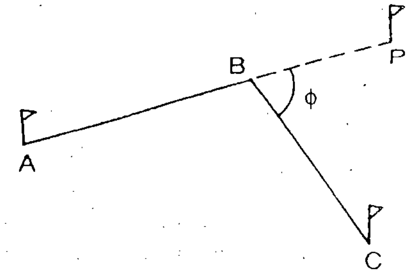


Fig. 9.11

6. Thus the deflection angle is doubled. The average of the two verniers is taken. One-half of this average value will give the correct value of the deflection angle.

The deflection angle is doubled in order to eliminate errors owing to wrong adjustment of the instrument, and those due to eccentricity of the centre.

9.11 MEASUREMENT OF MAGNETIC BEARING

Consider Fig. 9.12. Suppose the magnetic bearing of the line AB is to be measured. The procedure is as follow:

1. The theodolite is set up at A, and centred and levelled properly. Vernier A is set at 0° and vernier B at 180°. The upper clamp is fixed.
2. Now a trough compass or tubular compass is fixed on the left hand standard (A-frame) with a fixing screw. In some theodolites, a circular compass is provided over the vernier scale between the standards. However, the needle of the compass is released.

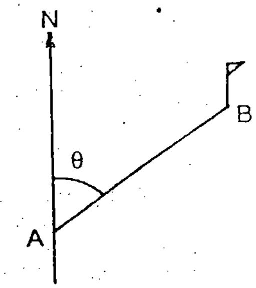


Fig. 9.12

3. By loosening the lower clamp, the telescope is rotated until it points to the north (i.e. the magnetic needle coincides with the '0–0' mark). At this time, the position of the telescope is said to be perfectly oriented along the magnetic meridian.
4. The lower clamp is fixed and the upper clamp loosened. Then by turning the telescope clockwise, the ranging rod at B is bisected with the help of the upper tangent screw.
5. The readings on both the verniers are taken. The mean of these readings is the magnetic bearing of AB.
6. The face of the instrument is changed, and the magnetic bearing of AB is measured in a similar manner.

7. The mean of the two observations will give the correct magnetic bearing of the line.

9.12 MEASURING HORIZONTAL DISTANCE BY STADIA METHOD

Consider Fig. 9.13. Suppose the horizontal distance between points A and B is to be determined by a theodolite. The following procedure is adopted:

1. The theodolite is set up at A. It is then centred and levelled properly. The vertical verniers (generally C and D) are set at the 'zero' of the vertical circle by turning the clamp screw and the tangent screw of the telescope (since the vertical circle is fixed to the telescope).

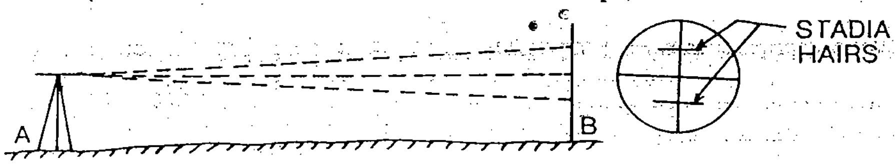


Fig. 9.13

2. The plate bubble is brought to the centre by turning the foot screws and the altitude bubble is centred by means of the clip screw (which is just below the altitude bubble). The line of collimation is made exactly horizontal in this manner.
3. The levelling staff is held at B, and the telescope is directed towards this point. By focussing the staff, the readings of the upper and lower stadia are noted. The difference between the two readings is calculated.
4. The required distance AB is obtained by multiplying this difference by the stadia constant, which is generally 100.

Example Suppose the readings of the upper and lower stadia are 3.255 and 1.365 respectively. Then

$$\text{Distance AB} = (3.255 - 1.365) \times 100 = 189 \text{ m}$$

9.13 RANGING AND EXTENDING A LINE

A. Ranging a Line

Ranging is the process of establishing intermediate points on a straight line between the terminal points. Let AB (Fig. 9.14) be the straight line on which intermediate points are to be fixed by theodolite. The procedure is as follows:

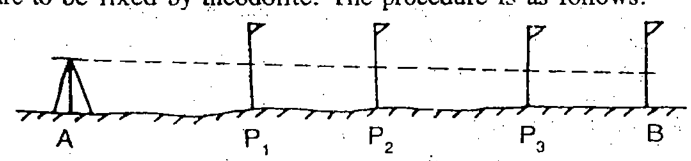


Fig. 9.14

1. The theodolite is centred over A and levelled properly. The upper clamp is fixed, and the lower clamp loosened. By turning the telescope the ranging rod at B is perfectly bisected with the help of the lower tangent screw. The lower clamp is now tightened.
2. Looking through the telescope, the observer directs the assistant to move the ranging rod to the left or right until it is on the straight line AB. Then the assistant fixes the ranging rod at P_1.
3. Then by lowering the telescope the observer finds the exact point P_1 on the ground which is marked by a nail or stake.
4. Similarly, the other points P_2, P_3, etc. are fixed and marked on the line.

B. Extending a Line

Consider Fig. 9.15. The following procedure is adopted if the line AB is to be extended:

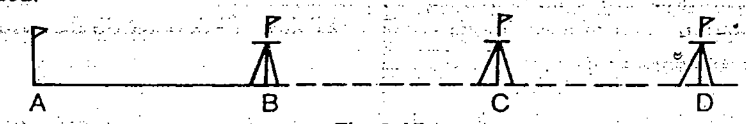


Fig. 9.15

1. The theodolite is set up at B, and centred and levelled perfectly.
2. The telescope is directed towards A and the ranging rod at A is perfectly bisected. The upper and lower clamps are fixed.
3. The telescope is transited. Looking through it, a ranging rod is fixed at C beyond the point B, along the line AB.
4. Now the theodolite is shifted and set up at C after removing the ranging rod. It is centred and levelled. Then a backsight reading is taken on B. The upper and lower clamps are fixed.
5. The telescope is transited and the next point D is fixed on the line by a ranging rod.
6. Similarly, other points are fixed on the line.

9.14 METHODS OF TRAVERSING

The following are the different methods of traversing:

1. Included angle method
2. Deflection angle method
3. Fast angle (or magnetic bearing) method

Note: Consider the traverse in anticlockwise direction as it is convenient to measure horizontal angles by theodolite.

A. Included Angle Method

This method is most suitable for closed traverse. The traverse may be taken in clockwise or anticlockwise order. Generally, a closed traverse is taken in the anticlockwise. In this method the bearing of the initial line is taken. After this, the included angles of the traverse are measured. These angles may be interior or exterior.

Procedure

1. Consider Fig. 9.16. The theodolite is set up and centred over A. The plate bubble is levelled. Vernier A is set at 0 and vernier B at 180°. The upper clamp is fixed.

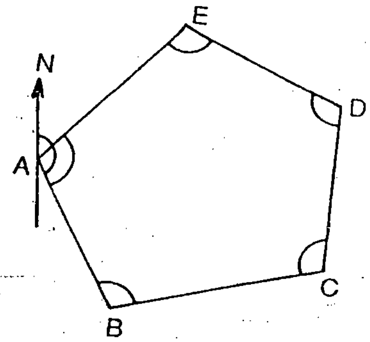


Fig. 9.16

2. The telescope is oriented along the north line with the help of the tubular compass fitted to the instrument. Then the magnetic bearing of AB is measured.
3. Again vernier A is set at 0° and the upper clamp is kept fixed.
4. The lower clamp is loosened and the ranging rod at E is bisected. Now, this clamp is tightened and the upper one opened. By turning the telescope clockwise, the ranging rod at B is bisected. The readings on the verniers are noted. ∠A is obtained in this fashion.
   The face of the instrument is changed and ∠A is measured once more. The mean of the two observations gives the correct value of ∠A.
5. Similarly, the other angles are measured by centring the theodolite at B, C, D and E.
   The arithmetical check is applied as follows:
   $$(2n - 4) \times 90° = \text{Sum of interior angles}$$
   If there is any discrepancy, the error is distributed among the angles.
6. For plotting the traverse, latitudes and departures of the traverse legs are calculated. The interior details are marked by applying the plane-table or transit-and-tape method (which will be described later).

B. Deflection Angle Method

This method is suitable for open traverse and is mostly employed in the survey of rivers, coast lines, roads, railways, etc.

Suppose an open traverse starts from A (Fig. 9.17). The following procedure is adopted:

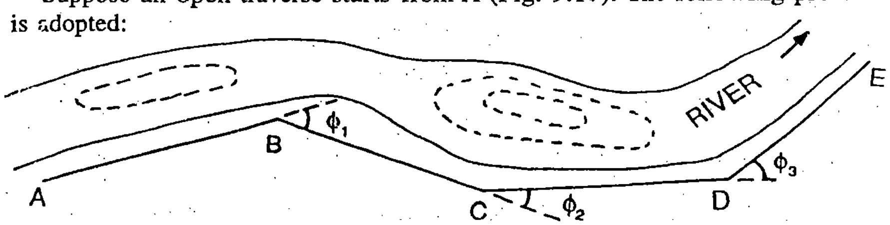


Fig. 9.17

1. The theodolite is set up at A, and then centred and levelled. After this, the bearing of the line AB is measured in the usual manner.
2. The theodolite is now shifted and centred over B. The plate bubble is levelled and vernier A set at 0°. Then a backsight is taken on A. The telescope is transited and by turning it clockwise the ranging rod at C is bisected. The vernier readings are taken. Then the deflection angle ϕ_1 is determined—it is the average value of the angles obtained from verniers A and B.
3. Similarly, the other deflection angles ϕ_2 and ϕ_3 are measured.
4. A field book is prepared in which the deflection angles and offsets are clearly noted.

C. Fast Needle Method

This method is used to measure the magnetic bearings and lengths of traverse legs. However, the angles between the lines are not measured. Suppose ABCDA is a closed traverse (Fig. 9.18). The following procedure is adopted:

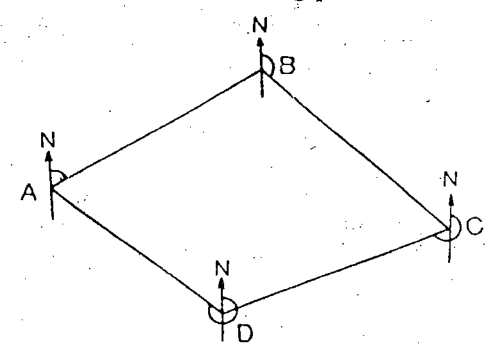


Fig. 9.18

1. The theodolite is set up at A. The vernier A is set at 0°. The telescope is oriented along the north line with the help of the trough compass or tubular compass fitted to the theodolite. The lower clamp is fixed.
2. The upper clamp is loosened and the ranging rod at B is bisected. The reading on vernier A gives the fore bearing of AB: say it is 30°. The backbearing of the line DA is also measured from A. Now the upper clamp is also fixed. The traverse is considered in clockwise direction.
3. The instrument is shifted and set up at B with vernier A fixed at the reading of 30°. The lower clamp is loosened and the ranging rod at A is bisected. The telescope is now transited. The upper clamp is then released and the ranging rod at C bisected. Now the reading on vernier A gives the bearing of BC: say it is 100°.
4. Again the instrument is shifted and set up at C with vernier A fixed at 100°.
5. The same process is repeated to get the fore bearing of CD.
6. Similarly, the fore bearings of the remaining sides are measured.
7. At the end of the traverse the FB and BB of DA should differ by 180°.

9.15 CHECK IN CLOSED AND OPEN TRAVERSE

A. Check in Closed Traverse

1. The sum of the measured interior angles should be equal to $(2N - 4) \times 90°$, and
2. The sum of the measured exterior angles should be equal to $(2N + 4) \times 90°$, where N is the number of sides.
3. The algebraic sum of the deflection angles should be equal to 360°, considering right-hand deflection to be positive and left-hand deflection negative.
4. The fore bearing and backbearing of the finishing line should differ by 180°.
5. The chaining of each line should be done twice, along opposite directions.
6. Check after computation: The sum of the northings should be equal to that of the southings, and the sum of the eastings should equal that of the westings.

B. Check in Open Traverse

In open traverse, measurements cannot be checked in the field. However, some field measurements are taken in order to ensure accuracy and determine the errors after plotting. The following are the field measurements taken for such check.

1. Tie line or cut-off line Suppose ABCDEF is an open traverse (Fig. 9.19). The cut-off lines AD and DF are suitably taken. The FB and BB of lines AD and DF are measured, and so are distances AD and DF. If after plotting the traverse, the distances, FB and BB of the cut-off lines tally with the field measurements, then the traverse is said to be correct.

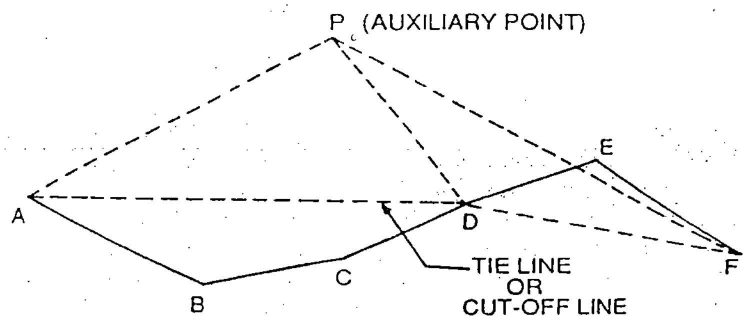


Fig. 9.19

2. Auxiliary point An auxiliary point P is suitably selected on one side of the traverse. Then magnetic bearings of this point are taken from A, D and F. If the traverse and plotting have been done accurately, then all these bearings must meet at P when plotted from the stations.

The traverse can also be checked by calculating the coordinates of the point P, considering ADP and DFP as closed figures.

If the coordinates of P, calculated from both sides, are equal, then the traverse may be assumed to be correct.

9.16 SOURCES OF ERROR IN THEODOLITE

A. Instrumental Errors

The following are the major causes of instrumental error:

1. Non-adjustment of plate bubble The axis of the plate bubble may not be perpendicular to vertical axis. So, when the plate levels are centred, the vertical axis may not be truly vertical. In such a case, the horizontal circle would be inclined and the angles will be measured in an inclined plane. This would cause an error in the angle measured.

This error may be eliminated by levelling the instrument with reference to the altitude bubble also.

2. Line of collimation not being perpendicular to horizontal axis In this case, a cone is formed when the telescope is revolved in the vertical plane, and this causes an error in the observation.

This error is eliminated by reading the angle from both faces (left and right), and taking the average of the readings.

3. Horizontal axis not being perpendicular to vertical axis If the horizontal axis is not perpendicular to the vertical axis, there is an angular error. This is eliminated by reading the angle from both faces.

4. Line of collimation not being parallel to axis of telescope If the line of collimation is not parallel to the axis of the telescope, there is an error in the observed vertical angle. This error is eliminated by taking readings from both faces.

5. Eccentricity of inner and outer axes This condition causes an error in vernier readings. This error is eliminated by taking readings from both verniers and considering the average of the readings.

6. Graduations not being uniform The error due to this condition is eliminated by measuring the angles several times on different parts of the circle.

7. Verniers being eccentric The zeroes of the verniers should be diametrically opposite to each other. When vernier A is set at 0°, vernier B should be at 180°. But in some cases, this condition may not exist.

This error is eliminated by reading both verniers and taking the average.

B. Personal Errors

1. The centring may not be done perfectly, due to carelessness.
2. The levelling may not be done carefully according to usual procedure.
3. If the clamp screws are not properly fixed, the instrument may slip.
4. The proper tangent screw may not be operated by mistake.
5. The focussing in order to avoid parallax may not be perfectly done.
6. The object or ranging rod may not be bisected accurately.
7. The verniers may not be set in proper place.
8. Errors would also result if the verniers are not read because of oversight.

C. Natural Errors

1. High temperature causes error due to irregular refraction.
2. High winds cause vibration in the instrument, and this may lead to wrong readings on the verniers.

9.17 CLOSING ERROR AND ITS LIMITATION

In a closed traverse the algebraic sum of latitudes must be equal to zero, and so should the algebraic sum of departures.

But due to the errors in field measurements of angles and lengths, sometimes the finishing point may not coincide with the starting point of a closed traverse. The distance by which a traverse fails to close is known as closing error or error of closure.

In Fig. 9.20, the traverse $ABCDA_1$ fails to close by a distance AA_1, which is the closing error of this traverse.

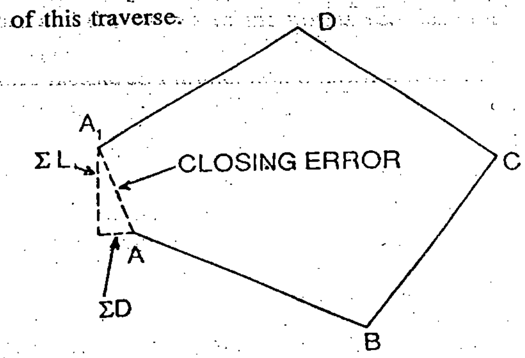


Fig. 9.20

$$\text{Closing error, } AA_1 = \sqrt{(\Sigma L)^2 + (\Sigma D)^2}$$

where L = latitude

and D = departure

$$\text{Relative closing error} = \frac{\text{closing error}}{\text{perimeter of traverse}}$$

$$\text{Permissible angular error} = \text{least count} \times \sqrt{N}$$

where N = number of sides

$$\tan\theta = \frac{\Sigma D}{\Sigma L}$$

where θ indicates the direction of closing error.

Table 9.5 *Permissible closing errors*

Traverse for	Permissible angular error	Permissible relative closing error
1. Land, roads, and railway surveys	$1' \times \sqrt{N}$	1 in 3000
2. City survey, important foundry survey	$30'' \times \sqrt{N}$	1 in 5000
3. Very important survey	$15'' \times \sqrt{N}$	1 in 10,000

N = number of sides of traverse.

9.18 COMPUTATION OF LATITUDE AND DEPARTURE

The theodolite traverse is not plotted according to interior angles or bearings. It is plotted by computing the latitudes and departures of the points (consecutive coordinates) and then finding the independent coordinates of the points.

The latitude of a line is the distance measured parallel to the North-South line and the departure of a line is measured parallel to the East-West line.

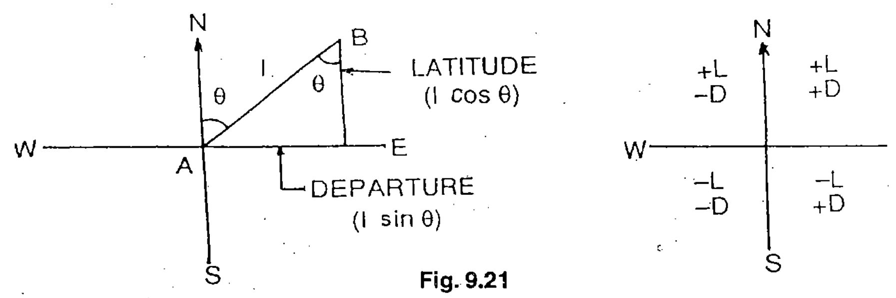


Fig. 9.21

The latitude and departure of lines are also expressed in the following ways:

Northing = latitude towards north = + L
Southing = latitude towards south = − L
Easting = departure towards east = + D
Westing = departure towards west = − D

Conversion of WCB to RB

WCB between	Corresponding RB	Quadrant
0° and 90°	RB = WCB	NE
90° and 180°	RB = 180° − WCB	SE
180° and 270°	RB = WCB − 180°	SW
270° and 360°	RB = 360° − WCB	NW

Table 9.6 *Table for computing latitude and departure*

Line	Length (L)	Reduced bearing (θ)	Latitude $(L \cos \theta)$	Departure $(L \sin \theta)$
AB	L	N θ E	+ L cos θ	+ L sin θ
BC	L	S θ E	− L cos θ	+ L sin θ
CD	L	S θ W	− L cos θ	− L sin θ
DA	L	N θ W	+ L cos θ	− L sin θ

Check for closed traverse

1. The algebraic sum of latitudes must be equal to zero.
2. The algebraic sum of departures must also be equal to zero.

Table 9.7

Line	Length	Reduced bearing	Consecutive Coordinates			
	(L)	(θ)	Northing (+)	Southing (−)	Easting (+)	Westing (−)
AB	L	N θ E	$L \cos \theta$		$L \sin \theta$	
BC	L	S θ E		$L \cos \theta$	$L \sin \theta$	
CD	L	S θ W		$L \cos \theta$		$L \sin \theta$
DA	L	N θ W	$L \cos \theta$			$L \sin \theta$

Check for closed traverse

1. Sum of northings = sum of southings
2. Sum of eastings = sum of westings

1. Consecutive coordinates The latitude and departure of a point calculated with reference to the preceding point for what are called consecutive coordinates.

2. Independent coordinates The coordinates of any point with respect to a common origin are said to be the independent coordinates of that point. The origin may be a station of the survey or a point entirely outside the traverse.

Consecutive coordinates may be positive or negative, depending upon the quadrant in which they lie.

In Gale's table the independent coordinates of all the points are made positive by suitably selecting the coordinates of the starting station of the traverse.

The coordinate of the starting station (i.e. origin) are assumed to be some positive values slightly greater than the maximum negative values of the latitudes and departures of the concerned traverse. Thus all the stations will ultimately come to the first quadrant when the coordinates of the traverse stations are correlated with the origin by computing the algebraic sum.

This method helps calculate the area of the traverse easily by the coordinate method, and also simplifies plotting of the traverse.

As for example, the coordinates of the starting station A are assumed to be (+ 150, + 50).

Thus all the stations are brought to the first quadrant. (as shown in the Table 9.8)

Table 9.8 *Independent coordinates of points of a traverse*

Point	Line	Consecutive coordinates			Independent coordinate			Remark
		Northing (+)	Southing (−)	Easting (+)	Westing (−)	Northing (+)	Easting (+)	
A	—	—	—	—	—	150.00	50.00	A is the
B	AB		115.00		40.00	35.00	10.00	starting
C	BC	5.00		50.00		40.00	60.00	point of
D	CD	80.00		25.00		120.00	85.00	traverse
A	DA	30.00			35.00	150.00	50.00	
Total		+ 115.00	− 115.00	+ 75.00	−75.00			

Max. negative latitude = − 115.00

Max. negative departure = − 40.00

9.19 BALANCING OF TRAVERSE

In case of closed traverse, the algebraic sum of latitudes must be equal to zero and that of departures must also be equal to zero in the ideal condition. In other words, the sum of the northings must equal that of the southings, and the sum of the eastings must be the same as that of the westings.

But in actual practice, some closing error is always found to exist while computing the latitude and departures of the traverse stations.

The total errors in latitude and departure are determined. These errors are then distributed among the traverse stations proportionately, according to the following rules.

1. Bowditch's rule By this rule, the total error (in latitude or departure) is distributed in proportion to the lengths of the traverse legs. This is the most common method of traverse adjustment.

(a) Correction to latitude of any side

$$= \frac{\text{length of that side}}{\text{perimeter of traverse}} \times \text{total error in latitude}$$

(b) Correction to departure of any side

$$= \frac{\text{length of that side}}{\text{perimeter of traverse}} \times \text{total error in departure}$$

2. Transit rule

(a) Correction to latitude of any side

$$= \frac{\text{latitude of that side}}{\text{arithmetical sum of all latitudes}} \times \text{total error in latitude}$$

(b) Correction to departure of any side

$$= \frac{\text{departure of that side}}{\text{arithmetical sum of all departures}} \times \text{total error in departures}$$

3. Third rule

(a) Correction to northing of any side

$$= \frac{\text{northing of that side}}{\text{sum of northings}} \times \frac{1}{2} \text{(total error in latitude)}$$

(b) Correction to southing of any side

$$= \frac{\text{southing of that side}}{\text{sum of southings}} \times \frac{1}{2} \text{(total error in latitude)}$$

(c) Correction to easting of any side

$$= \frac{\text{easting of that side}}{\text{sum of eastings}} \times \frac{1}{2} \text{(total error in departure)}$$

(d) Correction to westing of any side

$$= \frac{\text{westing of that side}}{\text{sum of westings}} \times \frac{1}{2} \text{(total error in departure)}$$

Note: If the error is positive, correction will be negative, and vice versa.

Example on Adjustment

1. Adjustment by Bowditch's rule

Table 9.9

Line	Length	Consecutive coordinate		Correction		Corrected consecutive coordinates	
		Latitude	Departure	Latitude	Departure	Latitude	Departure
AB	70.0	+ 21.500	– 65.450	+ 0.072	– 0.064	+ 21.572	– 65.514
BC	80.0	– 80.755	– 5.250	+ 0.083	– 0.073	– 80.672	– 5.323
CD	43.0	– 41.000	+ 13.550	+ 0.044	– 0.039	– 40.956	+ 13.511
DE	38.0	– 14.250	+ 35.150	+ 0.038	– 0.034	– 14.212	+ 35.116
EA	115.0	+ 114.150	+ 22.315	+ 0.118	– 0.105	+ 114.268	+ 22.210
Total	346.0	– 0.355	+ 0.315	+ 0.355	– 0.315	0	0
		Error		Correction		Adjusted	

Calculations of corrections

(a) Correction to latitude of:

$$AB = \frac{70}{346} \times 0.355 = +0.072$$

$$BC = \frac{80}{346} \times 0.355 = +0.083$$

$$CD = \frac{43}{346} \times 0.355 = +0.044$$

$$DE = \frac{38}{346} \times 0.355 = +0.038$$

$$EA = \frac{115}{346} \times 0.355 = +0.118$$

Total = + 0.355

(b) Correction to departure of:

$$AB = \frac{70}{346} \times (-0.315) = -0.064$$

$$BC = \frac{80}{346} \times (-0.315) = -0.073$$

$$CD = \frac{43}{346} \times (-0.315) = -0.039$$

$$DE = \frac{38}{346} \times (-0.315) = -0.034$$

$$EA = \frac{115}{346} \times (-0.315) = -0.105$$

Total = – 0.315

2. Adjustment by third rule

Table 9.10 (See on next page)

Calculation of corrections

$$\text{Correction to northing of AB} = \frac{21.500}{135.650} \times \frac{1}{2} \times 0.355 = +0.029$$

$$\text{Correction to southing of BC} = \frac{80.755}{136.005} \times \frac{1}{2} \times 0.355 = -0.106$$

$$\text{Correction to southing of CD} = \frac{41.000}{136.005} \times \frac{1}{2} \times 0.355 = -0.053$$

$$\text{Correction to southing of DE} = \frac{14.250}{136.005} \times \frac{1}{2} \times 0.355 = -0.019$$

$$\text{Correction of northing of EA} = \frac{114.150}{135.650} \times \frac{1}{2} \times 0.355 = +0.149$$

Total = + 0.000

$$\text{Correction to westing of AB} = \frac{65.450}{70.700} \times \frac{1}{2} \times 0.315 = +0.146$$

$$\text{Correction to westing of BC} = \frac{5.250}{70.700} \times \frac{1}{2} \times 0.315 = +0.012$$

$$\text{Correction to easting of CD} = \frac{13.550}{71.015} \times \frac{1}{2} \times 0.315 = -0.030$$

$$\text{Correction to easting of DE} = \frac{35.150}{71.015} \times \frac{1}{2} \times 0.315 = -0.078$$

$$\text{Correction to easting of EA} = \frac{22.315}{71.015} \times \frac{1}{2} \times 0.315 = -0.049$$

Total = + 0.001

Table 9.10

Line	Consecutive coordinates				Correction to				Corrected consecutive coordinates			
	Northing (+)	Southing (–)	Easting (+)	Westing (–)	Northing	Southing	Easting	Westing	Northing (+)	Southing (–)	East. (+)	Westing (–)
AB	21.500			65.45	+ 0.029			+ 0.146	21.529			65.596
BC		80.755		5.250		– 0.106		+ 0.012		80.650		5.262
CD		41.000	13.550			– 0.053	– 0.030			40.947	13.520	
DE		14.250	35.150			– 0.019	– 0.078			14.231	35.072	
EA	114.150		22.315		+ 0.149		– 0.049		114.299		22.266	
Total	135.650	136.005	71.015	70.700	+ 0.178	– 0.178	– 0.157	+ 0.158	135.828	135.828	70.858	70.858
	Error = – 0.355		Error = + 0.315						Error = 0.000		Error = 0.000	
	Correction = + 0.355		Correction = – 0.315									

Adjusted

9.20 CALCULATION OF TRAVERSE AREA

The area of a closed traverse may be calculated from:

1. The coordinates (x and y)
2. The latitude and double meridian distance
3. The departure and total latitudes.

A. Calculation of Area from Coordinates

The given consecutive coordinates of a traverse are converted into independent coordinates with reference to the coordinates of the most westerly station. Thus, the whole traverse is transferred to the first quadrant. In Fig. 9.22, A is the most westerly station.

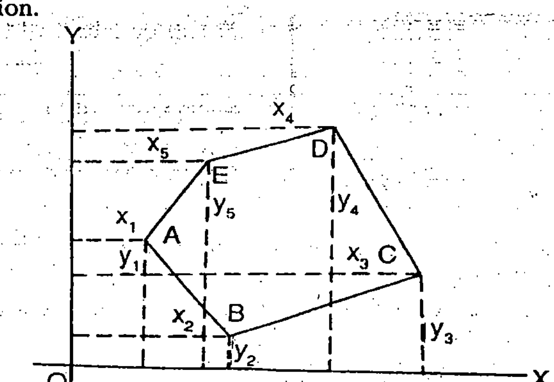


Fig. 9.22

Then, the coordinates are arranged in determinant form as follows:

$$\begin{matrix} y_1 & & y_2 & & y_3 & & y_4 & & y_5 & & y_1 \\ x_1 & & x_2 & & x_3 & & x_4 & & x_5 & & x_1 \end{matrix}$$

The sum of the products of coordinates joined by solid lines,

$$\Sigma P = (y_1x_2 + y_2x_3 + y_3x_4 + y_4x_5 + y_5x_1)$$

The sum of the products of coordinates joined by dotted lines,

$$\Sigma Q = (x_1y_2 + x_2y_3 + x_3y_4 + x_4y_5 + x_5y_1)$$

$\therefore$ Double area $= \Sigma P - \Sigma Q$

So, Required area $= \frac{1}{2} = (\Sigma P - \Sigma Q)$

Example for first method Find the area of the closed traverse having the following data, by the coordinate method.

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

Solution The consecutive coordinates are arranged in independent coordinate form as follows:

Station	Side	Consecutive coordinates		Independent coordinates	
		Latitude (y)	Departure (x)	Latitude (y)	Departure (x)
A				+ 200.00	+ 100.00
B	AB	+ 225.5	+ 120.5	+ 425.50	+ 220.50
C	BC	– 245.0	+ 210.0	+ 180.50	+ 430.50
D	CD	– 150.5	– 110.5	+ 30.00	+ 320.00
A	DA	+ 170.0	– 220.0	+ 200.00	+ 100.00

The independent coordinates of the most westerly station A are assumed to be (+ 200, + 100).

Thus, the independent coordinates of all the stations become positive (i.e. they come to the first quadrant).

The coordinates are now arranged in determinant form as follows:

$$\begin{matrix} 200.00 & & 425.50 & & 180.50 & & 30.00 & & 200.00 \\ & \times & & \times & & \times & & \times & \\ 100.00 & & 220.50 & & 430.50 & & 320.00 & & 100.00 \end{matrix}$$

Sum of products of coordinates joined by solid lines,

$$\Sigma P = (200.0 \times 220.5 + 425.5 \times 430.5 + 180.5 \times 320.0 + 30.0 \times 100.0)$$
$$= 288{,}037.75$$

Sum of products of coordinates joined by dotted lines,

$$\Sigma Q = (100.0 \times 425.5 + 220.5 \times 180.5 + 430.5 \times 30.0 + 320 \times 200.0)$$
$$= 159{,}265.25$$

$$\text{Required area} = \frac{1}{2}(\Sigma P - \Sigma Q) = \frac{1}{2}(288{,}037.75 - 159{,}265.25)$$
$$= 64{,}386.25 \text{ m}^2$$

B. Calculation of Area from Latitude and Double Meridian Distance (DMD)

Consider Fig. 9.23.

A is the most westerly station, and the reference meridian is assumed to pass through it.

Meridian distance (also called longitude) is the perpendicular distance between the midpoint of any line and the reference meridian.

The double meridian distance (DMD) or double longitude of a line is the distance equal to the sum of the meridian distances of the two ends of the line.

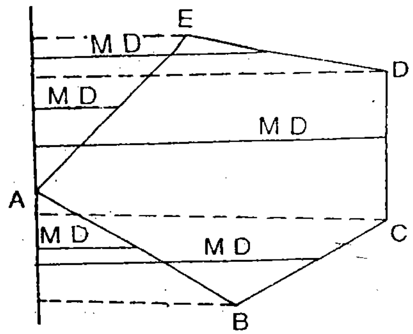


Fig. 9.23

Methods of finding DMD

1. DMD of first line = departure of first line.
2. DMD of second line = DMD of first line + departure of first line + departure of second line.
3. DMD of any succeeding line = DMD of preceding line + departure of preceding line + departure of line itself.
4. DMD of last line = departure of last line with opposite sign.

Procedure calculating area

1. Each DMD is multiplied by the latitude of that line.
2. The algebraic sum of these products is worked out.
3. This sum is equal to twice the area.
4. Half of this sum gives the required area of the traverse.

Points to remember

1. The reference meridian should pass through the most westerly station.
2. The signs of latitude and departure should always be taken into account.
3. A negative sign of the area does not carry any significance.

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

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

Solution

Calculation of DMD

DMD of AB = + 120.5
DMD of BC = + 120.5 + 120.5 + 210.0 = 451.0
DMD of CD = + 451.0 + 210.0 – 110.5 = + 550.5
DMD of DA = + 550.5 – 110.5 – 220.0 = + 220.0

The result is tabulated as follows:

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

(Negative sign neglected)

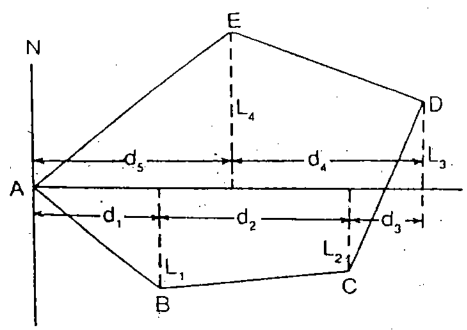
Twice area = algebraic sum

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

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

C. Calculation of Area from Departure and Total Latitude

Consider Fig. 9.24. A is the most westerly station, and the reference meridian is assumed to pass through it.



L_1, L_2......=LATITUDE

d_1, d_2....=DEPARTURE

Fig. 9.24

Procedure for calculating area

1. The total latitude (the latitude with respect to the reference point) of each station of the traverse is found out.
2. The algebraic sum of departures of the two lines meeting at a station is determined.
3. The total latitude is multiplied by the algebraic sum of departures, for each individual point.
4. The algebraic sum of this product gives twice the area.
5. Half of this sum gives the required area.

Example for third method Find the area of the closed traverse having the following data, by the departure and total latitude method:

Side	Latitude	Departures
AB	+ 225.5	+ 120.5
BC	– 245.0	+ 210.0
CD	– 150.5	– 110.5
DA	+ 170.0	– 220.0

Solution The latitudes of the stations are calculated with reference to station A,

Total latitude of B = + 225.5
Total latitude of C = + 225.5 – 245.0 = – 19.5
Total latitude of D = + 225.5 – 245.0 – 150.5 = – 170.0
Total latitude of A = + 225.5 – 245.0 – 150.5 + 170.0 = 0.0

Algebraic sum of departures at B = AB + BC = 120.5 + 210.0 = + 330.5
Algebraic sum of departures at C = BC + CD = + 210.0 – 110.5 = + 99.5
Algebraic sum of departures at D = CD + DA = – 110.5 – 220.0 = – 330.5
Algebraic sum of departures at A = DA + AB = – 220.0 + 120.5 = – 99.5

The result is tabulated as follows:

Side	Latitude	Departure	Station	Total latitude	Algebraic sum of adjoining departure	Double area (col. 5 × col. 6)	
						(+)	(–)
1	2	3	4	5	6	7	8
AB	+ 225.5	+ 120.5	B	+ 225.5	+ 330.5	74,527.75	
BC	– 245.0	+ 210.0	C	– 19.5	+ 99.5	—	1,940.25
CD	– 150.5	– 110.5	D	– 170.0	– 330.5	56,185.00	—
DA	+ 170.0	– 220.0	A	0.0	– 99.5	–	0
					Total	+ 130,712.75	– 1,940.25

Algebraic sum = + 128,772.50

Twice area = algebraic sum of column 7 and 8

Required area $= \frac{1}{2} \times 128{,}772.50 = 64{,}386.25 \text{ m}^2$

9.21 PROCEDURE FOR TRAVERSE SURVEY WITH THEODOLITE

A traverse survey with theodolite should be conducted according to the following steps:

Step 1 *Reconnaissance* The area to be surveyed is first thoroughly examined to decide the best possible way of starting the work. During reconnaissance, it should be remembered that the traverse legs should pass through fairly level ground. A traverse leg should be selected as the base line and there should be no magnetic substances near the stations on the base line. The magnetic bearing of the base line should be taken and there should be no local attraction. The traverse stations should cover the whole area to be surveyed.

Step 2 *Marking the stations* The traverse stations are marked on the ground by wooden pegs with nails on top. Reference sketches should be made, so that the stations may be located even if the station pegs are removed.

Step 3 *Measurement of interior angles* The interior angles of the traverse are measured by the method of measuring horizontal angles (as explained in Sec. 9.8). The lengths of the traverse legs are measured accurately, and so is the magnetic bearing of the starting line. All these measurements are entered in a traverse table, as shown in Table 9.11. (The table represents the records of the closed traverse ABCDEA in Fig. 9.25).

Step 4 *Measurement of magnetic bearing* Suppose the traverse leg AB is selected as a base line of the traverse ABCDEA. This base line should be free from local attraction. The theodolite is set up and centred over A. The plate bubble is levelled by foot screws. Vernier A is set at 0°, as usual. The tubular compass is attached to the left-hand standard. Then by loosening the lower clamp screw, the telescope is turned clockwise or anticlockwise until the needle of the compass coincides with '0–0' mark. The lower clamp is fixed and the upper one loosened. Then the ranging rod at B is bisected by turning the telescope clockwise. After that, the vernier reading is taken; this gives the magnetic bearing of AB.

The magnetic bearings of other traverse legs are calculated as shown in Step 6.

Step 5 *Correction of measured angles* (obtained from Table 9.11).

The observed angles are not correct.

$$\text{Correct sum of angles} = (2n - 4) \times 90° = (2 \times 5 - 4) \times 90° = 540°0'$$

So $\text{Error} = 539°58' - 540°0' = -0°2'$

$$\text{Correction per angle} = +\frac{120''}{5} = +24''$$

Table 9.11 *Traverse table*

Inst station	Object	Face	Angle	Reading on vernier A	Reading on vernier B	Angle on vernier A	Angle on vernier B	Mean angle of vernier	Mean angle of observation	Lengths of sides	WCB
A	E B	Left	∠A	—	—	—	—	—	∠A = 73°31′	AB = 66.6	FB of AB = 30°30′; BB of AB = 210°30′
A	E B	Right	∠A								
B	A C	Left	∠B						∠B = 107°42′	BC = 135.7	
B	A C	Right	∠B								
C	B D	Left	∠C						∠C = 187°8′	CD = 66.3	
C	B D	Right	∠C								

(Contd.)

Table 9.11 *(Contd.)*

Inst station	Object	Face	Angle	Reading on vernier A	Reading on vernier B	Angle on vernier A	Angle on vernier B	Mean angle of vernier	Mean angle of observation	Lengths of sides	WCB
D	C E	Left	∠D						∠D = 77°30′	DE = 76.6	
D	C E	Right	∠D								
E	D A	Left	∠E						∠E = 94°7′	EA = 214.3	
E	D A	Right	∠E								

Total = 539°58′

Notes: Detailed measurements are not shown here. Only the final angles are indicated for the preparation of Gale's table.

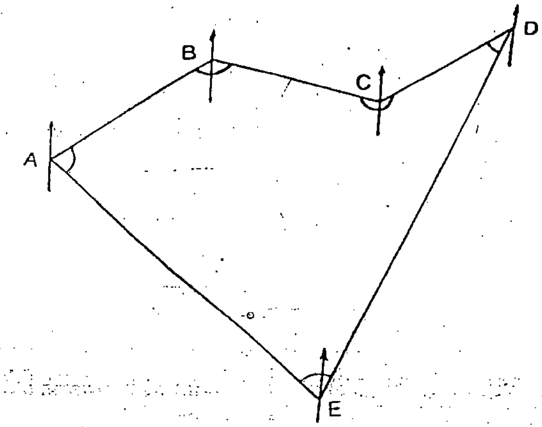


Fig. 9.25

Table 9.12 *Correction of angles*

Angle	Observed angle	Correction	Corrected angle
A	73°31′	+ 24″	73°31′24″
B	107°42′	+ 24″	107°42′24″
C	187°8′	+ 24″	187°8′24″
D	77°30′	+ 24″	77°30′24″
E	94°07′	+ 24″	94°07′24″
	Total = 539°58′		540°0′0″

Step 6 *Calculation of magnetic bearing* The magnetic bearings of all the traverse legs are calculated as follows (refer to Fig. 9.26):

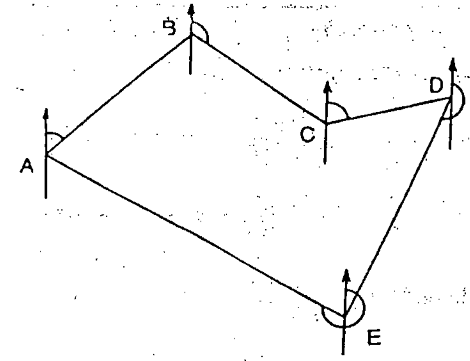


Fig. 9.26

FB of AB = 30°30′0″
FB of BC = BB of AB – ∠B
= 210°30′0″ – 107°42′24″ = 102°47′36″

FB of CD = BB of BC − ∠C
= 282°47′36″ − 187°8′24″ = 95°39′12″
FB of DE = BB of CD − ∠D
= 275°39′12″ − 77°30′24″ = 198°08′48″
FB of EA = BB of DE + Ex ∠E
= 18°08′48″ + 265°52′36″ = 284°1′24″
FB of AB = BB of EA − ∠A
= 104°1′24″ − 73°31′24″
= 30°30′0″ (checked)

The result is tabulated as follows:

Table 9.13

Line	Length (m)	WCB	RB
AB	66.6	30°30′0″	N 30°30′ E
BC	135.7	102°47′36″	S 77°12′24″ E
CD	66.3	95°39′12	S 84°20′48″ E
DE	76.6	198°8′48″	S 18°8′48″ W
EA	214.3	284°1′24″	N 75°58′36″ W

Step 7 Gale's table preparation from the recorded data, consecutive coordinates of all the stations are calculated. Then the independent coordinates of each station are worked out by assuming a suitable coordinate of the starting station or any other station so that all stations of the traverse come in the first quadrant, namely the NE quadrant. Then a table (known as Gale's table) is prepared showing all data recorded during the traverse. This table must be shown in the map just below the "heading". Table 9.14 shows the details of Gale's table.

The adjustment of the traverse is done according to the third rule. The details of calculation are shown in Table 9.14.

Detail of calculation for correction

Correction to latitudes (by third rule)

$$\text{Correction to northing of B} = \frac{57.383}{109.286} \times 0.016 = -0.008$$

$$\text{Correction to southing of C} = \frac{30.058}{109.254} \times 0.016 = +0.004$$

$$\text{Correction to southing of D} = \frac{6.411}{109.254} \times 0.016 = +0.001$$

$$\text{Correction to southing of E} = \frac{72.785}{109.254} \times 0.016 = +0.011$$

$$\text{Correction to northing of A} = \frac{51.903}{109.286} \times 0.016 = -0.008$$

Correction to departures (by third rule)

$$\text{Correction to easting of B} = \frac{33.799}{232.176} \times 0.19 = -0.028$$

$$\text{Correction to easting of C} = \frac{132.402}{232.176} \times 0.19 = -0.108$$

$$\text{Correction to easting of D} = \frac{65.975}{232.176} \times 0.19 = -0.054$$

$$\text{Correction to westing of E} = \frac{23.861}{231.796} \times 0.19 = +0.020$$

$$\text{Correction to westing of A} = \frac{207.955}{231.796} \times 0.19 = +0.170$$

Step 8 Location of interior details

The interior details may be located by:

1. The transit and tape method
2. The plane table method

1. Transit and tape method In this method, the theodolite is set up at the transit station (i.e. the traverse station) and the angle between the traverse leg and the object is measured. The distance of the object from the station is also measured and noted in a field book. There may be the following different cases.

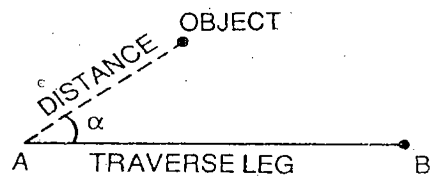


Fig. 9.27

(a) The object can be located by measuring the angle it makes with a station on the traverse leg, and the distance between it and the station (Fig. 9.27).

(b) The object can also be located by measuring the angle it makes with one station, and the distance between the object and the other station (Fig. 9.28).

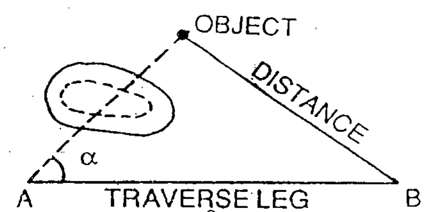


Fig. 9.28

(c) The object can be located by measuring the angles it makes with two stations (Fig. 9.29).

2. Plane table method The traverse is plotted on a drawing sheet according to Gale's table. The plotted traverse is then fixed on the plane table and the objects are located on the sheet by the radiation or intersection method after setting the plane table at each of the traverse stations. The procedure has been described in detail in chapter 4.

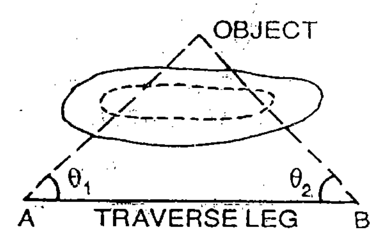


Fig. 9.29

Step 9 Plotting of the traverse

The following points should be taken care of while plotting the traverse:

1. A border line (about 20 mm from the edge of sheet) should drawn on all sides.
2. The heading should be written at the top. The heading may be "Theodolite Traverse Survey", "Theodolite Traverse Survey with Detailing by Transit and Tape", or "Theodolite traverse survey with Detailing by Plane Table".
3. Gale's table should be shown below the heading.
4. The north line should be marked on the right-hand top corner.
5. Considering the independent coordinates, the reference meridian should be so selected that the whole traverse may be accommodated in a suitable position on the sheet.
6. Conventional symbols should be shown on the right.
7. The scale should be drawn in a suitable position on the sheet.
8. The title block should be prepared on the right-hand bottom corner of the sheet.

A sample plot is shown in Fig. 9.30.

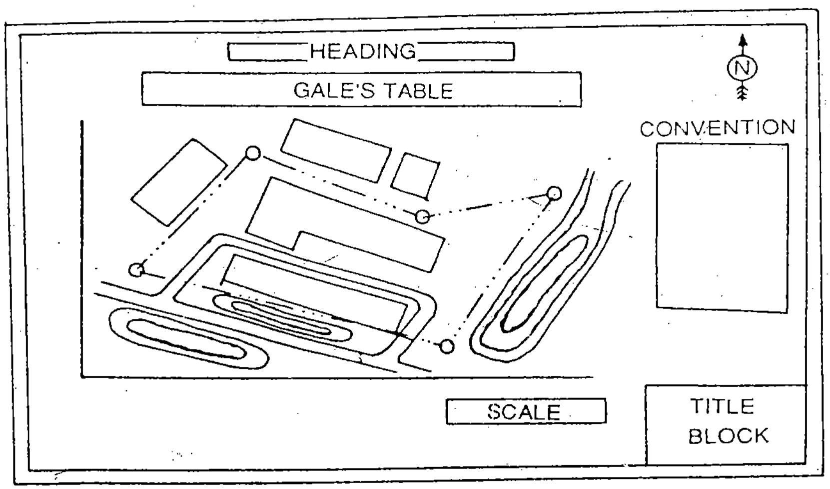


Fig. 9.30

9.22 WORKED-OUT PROBLEMS ON LATITUDE AND DEPARTURE, WITH INCOMPLETE DATA

Problem 1 The following records are obtained in a traverse survey, where the length and bearing of the last line were not recorded:

Line	Length (m)	Bearing
AB	75.50	30°24′
BC	180.50	110°36′
CD	60.25	210°30′
DA	?	?

Compute the length and bearing of line DA.

Solution Let L = length of DA

θ = bearing of DA

The calculations of latitudes and departures of the traverse are arranged in tubular form as follows:

Line	Length in m (L)	WCB	RB (θ)	Latitude ($L \cos \theta$)	Departure ($L \sin \theta$)
AB	75.50	30°24′	30°24′ NE	75.50 cos 30°24′ = + 65.12	75.50 sin 30°24′ = + 38.21
BC	180°50	110°36′	69°24′ SE	180°50 cos 69°24′ = − 63.51	180.50 sin 69°24′ = + 168.95
CD	60.25	210°30′	30°30′ SW	60.25 cos 30°− 30′ = − 51.91	60.25 sin 30°30′ = − 30.60
DA	L	—	θ	$L \cos \theta$	$L \sin \theta$

In a closed traverse, the algebraic sum of latitudes as also that of departures must be equal to zero.

So, $+ 65.12 - 63.51 - 51.91 + L \cos \theta = 0$ (1)

$L \cos \theta = 50.3$ (2)

Again $+ 38.21 + 168.95 - 30.60 + L \sin \theta = 0$ (3)

$L \sin \theta = -176.56$ (4)

Since the latitude of line DA is positive and the departure is negative, the line DA will be in the NW quadrant.

$$\frac{L \sin \theta}{L \cos \theta} = \frac{176.56}{50.30}$$ (here the signs of latitude and departure need not be considered)

or $\tan \theta = 3.5101$

$\theta = 74°5'$

Therefore, Bearing of DA = N 74°5′ W

$$\text{Distance DA} = \sqrt{(50.3)^2 + (176.56)^2}$$

$= 183.58$ m

Problem 2 An incomplete traverse table is obtained as follows:

Line	Length (m)	Bearing
AB	100.0	?
BC	80.5	140°30′
CD	60.0	220°30′
DA	?	310°15′

Calculate the length of DA and bearing of AB.

Solution Let L = length of DA
θ = bearing of AB

The calculations of latitudes and departures of the traverse are arranged in tabular form as follows:

Line	Length	WCB	RB	Latitude	Departure
AB	100.0	—	θ	100 cos θ	100 sin θ
BC	80.00	140°30′	39°30′ SE	80.5 cos 39°30′ = – 62.12	80.5 sin 39°30′ = + 51.20
CD	60.0	220°30′	40°30′ SW	60.0 cos 40°30′ = – 45.62	60.0 sin 40°30′ = – 38.97
DA	L	310°15′	49°45′ NW	L cos 49°45′ = + 0.646 L	L sin 49°45′ = – 0.763 L

In a closed traverse, the algebraic sum of latitudes must be equal to zero, and so should that of the departures.

So, $$100\cos\theta - 62.12 - 45.62 + 0.646L = 0$$

$$100\cos\theta = 107.74 - 0.646L \qquad (1)$$

Again, $$100\sin\theta + 51.20 - 38.97 - 0.763L = 0$$

$$100\sin\theta = 0.763L - 12.23 \qquad (2)$$

Squaring and adding Eqs (1) and (2), we get

$$(100)^2 = (107.74 - 0.646L)^2 + (0.763L - 12.23)2$$
$$= 11{,}607.9 - 139.2L + 0.42L^2 + 0.58L^2 - 18.66L + 149.6$$
$$= 11{,}757.5 - 157.86L + L^2$$

or $$L^2 - 157.86L + 1{,}757.5 = 0$$

$$\therefore \quad L = \frac{157.86 \pm \sqrt{(157.86)^2 - 4 \times 1{,}757.5}}{2}$$

$$= \frac{157.86 \pm 133.75}{2} = 145.8 \text{ m or } 12.04 \text{ m}$$

When length of DA, L = 145.8 m

From (1), $100\cos\theta = 107.74 - 0.646 \times 145.8 = 13.55$
or $\cos\theta = 0.1355$ (+ve)
From (2), $100\sin\theta = 0.763 \times 145.8 - 12.23 = 99.01$
or $\sin\theta = 0.9901$ (+ve)

Since both cos θ and sin θ are positive, the line AB lies in the NE quadrant.

$$\tan\theta = \frac{0.9901}{0.1355} = 7.3070$$

$$\theta = 82°12'$$

So, bearing of AB = N 82°12′ E

When length of DA, L = 12.04 m

From (1), $100\cos\theta = 107.74 - 0.646 \times 12.04 = 99.96$
$\cos\theta = 0.9996$ (+ve)
From (2), $100\sin\theta = 0.763 \times 12.04 - 12.23 = -3.04$
$\sin\theta^\circ = -0.0304$ (–ve)

Since cos θ is positive and sin θ negative, the line AB lies in the NW quadrant.

$$\tan\theta = \frac{0.0304}{0.9996} = 0.0304$$

$$\theta^\circ = 1°44'$$

So, bearing of AB = N 1°44′ W

Note: If the value of the length calculated by solving the quadratic equation turns out to be negative, the value should be ignored.

Problem 3 The record of a closed traverse is given below, with two distances missing.

Line	Length (m)	Bearing
AB	100.5	N 30°30′ E
BC	?	S 45°0′ E
CD	75.0	S 40°30′ W
DE	50.5	S 60°0′ W
EA	?	N 40°15′ W

Calculate the lengths of BC and EA.

Solution Let l_1 = length of BC
l_2 = length of EA

The calculations of latitudes and departures of the lines are tabulated as follows:

)	Departure (L in θ)
s 30°30′ 9	100.5 sin 30°30′ = + 51.00
5° 07 l_1	l_1 sin 45° = + 0.707 l_1
40°30′ 03	75.0 sin 40°30′ = − 48.71
s 60°0′ 25	50.5 sin 60°0′ = − 43.73
5°15′ 63 l_2	l_2 sin 40°15′ = − 0.646 l_2

as also that of departures must

).763 l_2 = 0

0.707l_1 = − 4.31 (1)

0.646l_2 = 0

- 0.646l_2 = 41.44 (2)

m

7.35 + 4.31

for a traverse survey, where the
1. Point A is 50.0 m from P, and

Bearing
N 30°15′ E
S 40°30′ E
S 60°30′ W

Table 9.14 *Gale's table*

Instrument Station	Observed angle	Correction	Corrected angle	Line	Length	WCB	Reduced bearing	Point	Consecutive coordinates: Latitude ($L\cos\theta$) Northing (+)	Latitude Southing (−)	Departure ($L\sin\theta$) Easting (+)	Departure Westing (−)	Correction: Northing	Southing	Easting	Westing	Corrected Consecutive coordinates: Northing (+)	Southing (−)	Easting (+)	Westing (−)	Independent coordinates: Northing (+)	Easting (+)	Remarks
A	73°31′0″	+ 24″	73°31′24″					A	—	—	—	—	—	—	—	—	—	—	—	—	100.000	200.000	A is the starting point of the traverse
B	107°42′0″	+ 24″	107°42′24″	AB	66.6	30°30′	30°30′ N E	B	57.383		33.799				− 0.028		57.375		33.771		157.375	233.771	
C	187°8′0″	+ 24″	187°8′24″	BC	135.7	102°47′36″	77°12′24″ S E	C		30.058	132.402		− 0.008	+ 0.004	− 0.108			30.062	132.294		127.313	366.065	
D	77°30′0″	+ 24″	77°30′24″	CD	66.3	95°39′12″	84°20′48″ S E	D		6.411	65.975			+ 0.001	− 0.054			6.412	65.921		120.901	431.986	
E	94°7′0″	+ 24″	94°7′24″	DE	76.6	198°8′48″	18°8′48″ S W	E		72.785		23.861		+ 0.011		+ 0.020		72.796		23.881	48.105	408.105	
Total	539°58′0″	+ 0°2′0″	540°0′0″	EA	214.3	284°1′24″	75°58′36″ N W	A	51.903			207.935	− 0.008			+ 0.170	51.895			208.105	100.000	200.000	
							Total Error		+ 0.032		+ 0.380		− 0.016	+ 0.006	− 0.19	+ 0.19	109.27	109.27	231.986	231.986			
							Total Correction		− 0.032		− 0.380		Adjusted				0.000		0.000		Error nil		

Line	Length in m (L)	RB (θ)	Latitude (L cos θ)	Departure (L in θ)
AB	100.5	30°30′ NE	100.5 cos 30°30′ = + 86.59	100.5 sin 30°30′ = + 51.00
BC	l_1	45°0′ SE	l_1 cos 45° = − 0.707 l_1	l_1 sin 45° = + 0.707 l_1
CD	75.0	40°30′ SW	75.0 cos 40°30′ = − 57.03	75.0 sin 40°30′ = − 48.71
DE	50.5	60°0′ SW	50.5 cos 60°0′ = − 25.25	50.5 sin 60°0′ = − 43.73
EA	l_2	40°15′ NW	l_2 cos 45°15′ = + 0.763 l_2	l_2 sin 40°15′ = − 0.646 l_2

In a closed traverse, the algebraic sum of latitudes as also that of departures must be equal to zero.

So,

$$+86.59 - 0.707\ l_1 - 57.03 - 25.25 + 0.763\ l_2 = 0$$

or

$$0.763 l_2 - 0.707 l_1 = -4.31 \quad (1)$$

Again,

$$+51.00 + 0.707 l_1 - 48.71 - 43.73 - 0.646 l_2 = 0$$

or

$$0.707 l_1 - 0.646 l_2 = 41.44 \quad (2)$$

Adding Eqs (1) and (2), $0.117 l_2 = 37.13$

∴ $l_2 = 317.35$ m

Putting the value of l_2 in Eq. (1),

$$0.707 l_1 = 0.763 \times 317.35 + 4.31$$

$$l_1 = 348.58 \text{ m}$$

Problem 4 The following particulars are given for a traverse survey, where the length of the line AB is required to be measured. Point A is 50.0 m from P, and B is 75.0 m from R.

Line	Length (m)	Bearing
PQ	125.5	N 30°15′ E
QR	80.25	S 40°30′ E
RS	150.75	S 60°30′ W

Table 9.14 *Gale's table*

Instrument Station	Observed angle	Correction	Corrected angle	Line	Length	WCB	Reduced bearing	Point	Consecutive coordinates: Latitude (L cos θ) Northing (+)	Latitude Southing (−)	Departure (L sin θ) Easting (+)	Departure Westing (−)	Correction: Northing	Southing	Easting	Westing	Corrected Consecutive coordinates: Northing (+)	Southing (−)	Easting (+)	Westing (−)	Independent coordinates: Northing (+)	Easting (+)	Remarks
A	73°31′0″	+ 24″	73°31′24″					A	—	—	—	—	—	—	—	—	—	—	—	—	100.000	200.000	A is the starting point of the traverse
B	107°42′0″	+ 24″	107°42′24″	AB	66.6	30°30′	30°30′ N E	B	57.383		33.799				− 0.028		57.375		33.771		157.375	233.771	
C	187°8′0″	+ 24″	187°8′24″	BC	135.7	102°47′36″	77°12′24″ S E	C		30.058	132.402		− 0.008	+ 0.004	− 0.108			30.062	132.294		127.313	366.065	
	′		4″			12″	48″				5			001	054			2	921		0.901	1.986	

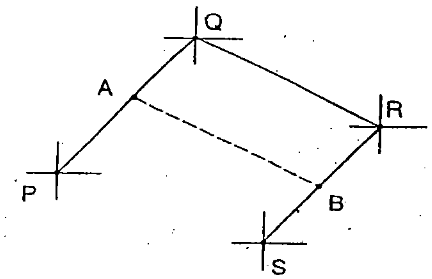


Fig. P.9.1

Solution

The given traverse is rearranged as a closed traverse AQRBA.

Let l = length of BA

θ = bearing of BA

The calculations of latitudes and departures are tabulated as follows:

Line	Length (m)	RB	Latitude	Departure
AQ	75.5	30°15′ NE	75.5 cos 30°15′ = + 65.22	75.5 sin 30°15′ = + 38.03
QR	80.25	40°30′ SE	80°25 cos 40°30′ = – 61.02	80.25 sin 40°30′ = + 52.12
RB	75.0	60°30′ SW	75.0 cos 60°30′ = – 36.93	75.0 sin 60°30′ = – 65.28
BA	l	θ	$l \cos \theta$	$l \sin \theta$

In a closed traverse, the algebraic sum of latitudes and departures must be equal to zero.

So, $$+ 65.22 - 61.02 - 36.93 + l \cos \theta = 0$$

$$l \cos \theta = 32.73 \quad (1)$$

Again, $$+ 38.03 + 52.12 - 65.28 + l \sin \theta = 0$$

$$l \sin \theta^\circ = - 24.87 \quad (2)$$

Squaring and adding Eqs. (1) and (2),

$$l^2 = (32.73)^2 + (24.87)^2$$

∴ $$l = \sqrt{(32.73)^2 + (24.87)^2}$$

or $$l = 41.11 \text{ m}$$

So, length AB = 41.11 m

Problem 5 The following observations were taken from stations P and Q.

Line	Length (m)	Bearings
PA	125.0	S 60°30′ W
PQ	200.0	N 30°30′ E
QB	150.5	N 50°15′ W

Calculate the length and bearing of AB, and also the angles ∠PAB and ∠QBA.

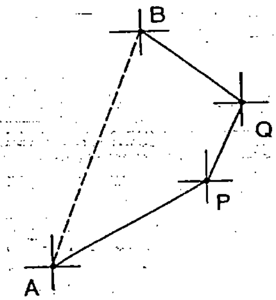


Fig. P.9.2

Solution

Let the figure be rearranged as a closed traverse BQPAB and tabulated as follows.

The reduced bearings of BQ and QP will be numerically equal to that of QB and PQ, but their respective quadrants will be opposite. That is

RB of BQ = S 50°15′ E
RB of QP = S 30°30′ W

Let l = length of AB
θ = bearing of AB

In a closed traverse, the algebraic sum of latitude and departures must be equal to zero.

So,
$$-96.24 - 172.32 - 61.55 + l\cos\theta = 0$$
$$l\cos\theta = 330.11 \quad (1)$$
$$+115.71 - 101.50 - 108.79 + l\sin\theta = 0$$
$$l\sin\theta = 94.58 \quad (2)$$

Since the latitude and departure are both positive, the line AB lies in the NE quadrant.

$$\tan\theta = \frac{94.58}{330.11}$$

∴ $\theta = 15°59'$

So, Bearing of AB = N 15°59′ E

Line	Length in m (L)	RB (θ)	Latitude ($L\cos\theta$)	Departure ($L\sin\theta$)
BQ	150.5	50°15′ SE	150.5 cos 50°15′ = – 96.24	150.5 sin 50°15′ = + 115.71
QP	200.0	30°30′ SW	200.0 cos 30°30′ = – 172.32	200.0 sin 30°30′ = – 101.50
PA	125.0	60°30′ SW	125.0 cos 60°30′ = – 61.55	125.0 sin 60°30′ = – 108.79
AB	l	θ	$l\cos\theta$	$l\sin\theta$

$$\text{Length AB} = \sqrt{(330.11)^2 + (94.58)^2}$$
$$= 343.39 \text{ m}$$

$$\angle PAB = \text{BB of AP} - \text{FB of AB}$$
$$= 60°30' - 15°59' = 44°31'$$

$$\angle QBA = \text{FB of BQ} + \text{BB of AB}$$
$$= 50°15' + 15°59' = 66°14'$$

Problem 6 In an open traverse ABCDE, it is required to fix the midpoint of the line joining A and E. Find the length and bearing of that point from the station C, when the records of the traverse are as follows:

Line	Length (m)	Bearing
AB	130.5	N 20°30′ E
BC	215.0	N 60°15′ E
CD	155.5	S 30°30′ E
DE	120.0	N 80°30′ E

Solution

The figure ABCDEA is considered as a closed traverse.

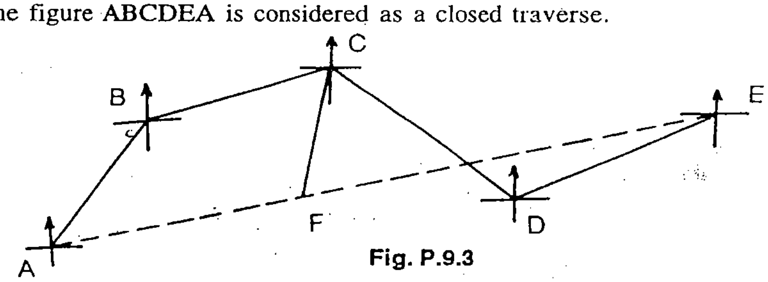


Fig. P.9.3

Let

l = length of EA
θ = bearing of EA

The calculations are tabulated as follows:

Line	Length in (m) (L)	RB (θ)	Latitude $(L \cos \theta)$	Departure $(L \sin \theta)$
AB	130.5	20°30′ NE	130.5 cos 20°30′ = + 122.23	130.5 sin 20°30′ = + 45.70
BC	215.0	60°15′ NE	215.0 cos 60°15′ = + 106.68	215.0 sin 60°15′ = + 186.66
CD	155.5	30°30′ SE	155.5 cos 30°30′ = − 133.98	155.5 sin 30°30′ = + 78.92
DE	120.0	80°30′ NE	120.0 cos 80°30′ = + 9.81	120.0 sin 80°30′ = + 118.35
EA	l	θ	$l \cos \theta$	$l \sin \theta$

In a closed traverse, the algebraic sum of latitudes and departures must be equal to zero.

So, $$+122.23 + 106.68 - 133.98 + 19.81 + l \cos \theta = 0$$
$$l \cos \theta = -114.74 \quad (1)$$

and $$+45.70 + 186.66 + 78.92 + 118.35 + l \sin \theta = 0$$
$$l \sin \theta = -429.63 \quad (2)$$

Since the latitude and departure are both negative, the line EA lies in the SW quadrant.

$$\tan \theta = \frac{429.63}{114.74}$$

∴ $$\theta = 75°3'$$

That is, Bearing of EA = S 75°3′ W

and $$\text{Length of EA, } l = \sqrt{(114.74)^2 + (429.63)^2}$$
$$= 444.68 \text{ m}$$

Let us consider the point F as the mid-point of AE.

So, FA = 222.34 m

Bearing of FA = S 75°3′ W

(which is the same as that of EA).

Now, considering ABCFA as a closed traverse, the length of CF is taken as l_1 and the bearing as θ_1.

The calculations are tabulated as follows:

Line	Length	RB	Latitude	Departure
AB	130.5	20°30′ NE	130.5 cos 20°30′ = + 122.24	130.5 sin 20°30′ = + 45.70
BC	215.0	60°15′ NE	215.0 cos 60°15′ = + 106.68	215.0 sin 60°15′ = + 186.66
CF	l_1	θ_1	$l_1 \cos \theta_1$	$l_1 \sin \theta_1$
FA	222.34	75°3′ SW	222.34 cos 75°3′ = − 57.36	222.34 sin 75°3′ = − 214.81

As Σ Latitude = 0

and Σ Departure = 0

$$+122.24 + 106.68 + l_1 \cos \theta_1 - 57.36 = 0$$
$$l_1 \cos \theta_1 = -171.56 \quad (1)$$
$$+45.70 + 186.66 + l_1 \sin \theta_1 - 214.81 = 0$$
$$l_1 \sin \theta_1 = -17.55 \quad (2)$$
$$l_1^2 = (171.56)^2 = (17.55)^2 \quad \text{[squaring and adding Eqs (1) and (2)]}$$

∴ $$l_1 = 172.46 \text{ m}$$

Since latitude and departure are both negative, the line CF lies in the SW quadrant.

$$\tan \theta_1 = \frac{17.55}{171.56} = 0.1022$$
$$\theta_1 = 5°50'$$

So, Bearing of CF = S 5°50′ W

Therefore, the distance of F from C is 172.48 m and the bearing of F from C is S 5°50′ W.

Problem 7 The following data refer to a closed traverse. Compute the missing quantities.

Line	Length (m)	Bearing
AB	725	S 60°00′ E
BC	1050	?
CD	1250	?
DE	950	S 55°30′ W
EA	575	S 02°45′ W

(AMIE, Summer 1990)

Solution The calculations of latitude and departures of the traverse tabulated as follows:

Line	Length in (m) (L)	Bearing (θ)	Latitude in (m) (L cos θ)	Departure in (m) (L sin θ)
AB	725	S 60°00′ E	725 cos 60°00′ = – 362.50	+ 725 sin 60°00′ = + 627.87
BC	1,050	θ_1	1,050 cos θ_1	1,050 sin θ_1
CD	1,250	θ_2	1,250 cos θ_2	1,250 sin θ_2
DE	950	S 55°30′ W	– 950 cos 55°30′ = – 538.08	– 950 sin 55°30′ = – 782.92
EA	575	S 02°45′ W	– 575 cos 02°45′ = – 574.34	– 575 sin 2°45′ = – 27.59

In a closed traverse, the algebraic sum of latitudes as also that of departures must be equal to zero.

$$-362.50 + 1{,}050 \cos\theta_1 + 1{,}250 \cos\theta_2 - 538.08 - 574.34 = 0$$

or $$1{,}050 \cos\theta_1 + 1{,}250 \cos\theta_2 = 1{,}474.92$$

or $$\cos\theta_1 + 1.19 \cos\theta_2 = 1.405 \qquad (1)$$

and $$627.87 + 1{,}050 \sin\theta_1 + 1{,}250 \sin\theta_2 - 782.92 - 27.59 = 0$$

or $$1{,}050 \sin\theta_1 + 1{,}250 \sin\theta_2 = 182.64$$

or $$\sin\theta_1 + 1.19 \sin\theta_2 = 0.174 \qquad (2)$$

Squaring and adding Eqs (1) and (2), we get

$$(\cos\theta_1 + 1.19\cos\theta_2)^2 + (\sin\theta_1 + 1.19\sin\theta_2)^2 = (1.405)^2 + (0.174)^2$$

or $$\cos^2\theta_1 + \sin^2\theta_1 + 2 \times 1.19(\cos\theta_1\cos\theta_2 + \sin\theta_1\sin\theta_2) + (1.19)^2(\cos^2\theta_2 + \sin^2\theta_2) = 2.004$$

or $$1 + 2.38\cos(\theta_1 - \theta_2) + 1.42 \times 1 = 2.004$$

or $$2.38\cos(\theta_1 - \theta_2) = -0.416$$

or $$\cos(\theta_1 - \theta_2) = -0.175 = -\cos 79.92°$$

or $$\cos(\theta_1 - \theta_2) = \cos 100.08° \qquad \because \ -\cos\theta = \cos(180° - \theta)$$

$$\theta_1 - \theta_2 = 100.08°$$

$$\theta_1 = 100.08° + \theta_2$$

Putting $\theta_1 = 100.08 + \theta_2$ in Eq. (1), we get

$$\cos(100.08° + \theta_2) + 1.19\cos\theta_2 = 1.405$$

or $$\cos 100.08° \cos\theta_2 - \sin 100.08° \sin\theta_2 + 1.19\cos\theta_2 = 1.405$$

or $$-0.175\cos\theta_2 - 0.984\sin\theta_2 + 1.19\cos\theta_2 = 1.405$$

or $$1.015\cos\theta_2 - 0.984\sin\theta_2 = 1.405$$
$$[\because \ \sin\theta = \sin(180° - \theta)]$$

or $$\cos\theta_2 - 0.969\sin\theta_2 = 1.384 \qquad (3)$$

Again, substituting the value of θ_1 in Eq. (2),

we get, $$\sin(100.08 + \theta_2) + 1.19\sin\theta_2 = 0.174$$

or $$\sin 100.08\cos\theta_2 + \cos 100.08\sin\theta_2 + 1.19\sin\theta_2 = 0.174$$

or $$0.984\cos\theta_2 - 0.175\sin\theta_2 + 1.19\sin\theta_2 = 0.174$$

or $$0.984\cos\theta_2 + 1.015\sin\theta_2 = 0.174$$

or $$\cos\theta_2 + 1.031\sin\theta_2 = 0.177 \qquad (4)$$

Subtracting Eq. (3) from (4), we get

$$\cos\theta_2 + 1.031\sin\theta_2 - (\cos\theta_2 - 0.969\sin\theta_2) = 0.174 - 1.384$$

or $$2\sin\theta_2 = -1.207$$

or $$\sin\theta_2 = -0.6035 \qquad (5)$$

Substituting the value of $\sin\theta_2$ in Eq. (3), we get

$$\cos\theta_2 - 0.969 \times (-0.6035) = 1.384$$

or $$\cos\theta_2 = 0.7992 \qquad (6)$$

From Eq. (5) and (6), we get $\tan\theta_2 = 0.7551$

$$\theta_2 = 37°3'$$

Since the value of $\cos\theta_2$ is positive and that of $\sin\theta_2$ is negative. The line CD lies in the NW quadrant.

So, Bearing of CD = N 37°3′ W

Again, substituting the value of $\cos\theta_2$ in Eq. (1) and that of $\sin\theta_2$ in Eq. (2), we get

$$\cos\theta_1 + 1.19 \times 0.7992 = 1.405$$

or $$\cos\theta_1 = 0.4539 \qquad (7)$$

and $$\sin\theta_1 + 1.19 \times (-06035) = 0.174$$

or $$\sin\theta_1 = 0.8922 \qquad (8)$$

From Eqs (7) and (8), we get

$$\tan\theta_1 = 1.9655$$
$$\theta_1 = 63°2'$$

The values of both $\cos\theta_1$ and $\sin\theta_1$ are positive.

The line BC lies in the NE quadrant.

Bearing of BC = N 63°2′ E

9.23 PERMANENT ADJUSTMENT OF THEODOLITE

A theodolite consists of several fundamental lines. In order the readings to be accurate, certain desired relationships must exist between the fundamental lines of the instrument. But due to improper handling or excessive use, this relationship may be disturbed and hence the readings from the theodolite may lead to erroneous results.

For rectifying a disturbed relationship, some procedures, termed permanent adjustments, are adopted.

The fundamental lines of a theodolite are:

1. The vertical axis
2. The axis of the plate level (i.e. the plate bubble)
3. The line of collimation
4. The horizontal axis or trunnion axis
5. The bubble line of the altitude level (the axis of the altitude bubble).

The desired relationships between the fundamental lines are as follows:

1. The axis of the plate level must be perpendicular to the vertical axis.
2. The line of collimation should coincide with the optical axis of the telescope, and should also be perpendicular to the vertical axis.
3. The horizontal axis must be perpendicular to the vertical axis.
4. The axis of the telescope must be parallel to the line of collimation.
5. The line of collimation must be perpendicular to the horizontal axis, and the vertical circle should read zero when the line of collimation is horizontal.

First adjustment To make the axis of the plate level perpendicular to the vertical axis, the following procedure is adopted prior to the first adjustment:

1. The theodolite is set up on firm ground with its legs well apart, and firmly fixed on the ground.
2. The plate bubble is made parallel to any pair of foot screws, and brought to the centre of its run by turning the concerned foot screws.
3. The bubble is turned through 90° and then brought to the centre by turning the third foot screw.
4. The process is repeated several times until the bubble is perfectly centred in these two positions.
5. The bubble is turned through 180° about the vertical axis.
6. If the bubble still remains in the central position, the axis of the bubble is perpendicular to the vertical axis which may then be assumed to be truly vertical.
7. If the bubble does not remain in the central position after step (5), the amount of deviation is noted, say it is $2n$ divisions.

Adjustment

1. Half of the total deviation (i.e. n divisions) is adjusted by means of the capstan-headed nut provided just below the bubble tube.
2. The remaining half (i.e. n divisions is adjusted by turning the concerned foot screws.

3. The process is repeated several times until the bubble remains in the central position for any direction of the bubble tube.

Second adjustment To make the line of collimation coincide with the optical axis of the telescope, first the horizontal hair and then the vertical hair are adjusted.

1. Adjustment of horizontal hair (Fig. 9.31)

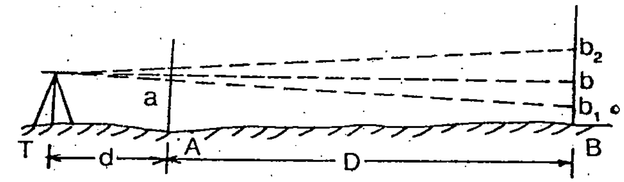


Fig. 9.31

(a) Three pegs are driven into the ground at T, A and B, a known distance apart.
(b) The theodolite is set up at T and after proper adjustment staff readings are taken on A and B. Suppose the readings are Aa and Bb_1.
(c) By transisting the theodolite the staff readings are taken on A and B.
(d) If the readings of the second observation tallies with those of the first the horizontal hair is in adjustment.
(e) If the second observation gives a new reading, say Bb_2, then the horizontal hair requires adjustment.

Adjustment

(a) The mean of the two staff readings obtained from the two observations is calculated. Let it be Bb.
(b) By turning the vertical diaphragm screw, the horizontal hair is brought to the reading Bb.
(c) The process is repeated for several times until the horizontal hair is perfect in all conditions.

2. Adjustment of Vertical Hair (Fig. 9.32)

To make the line of collimation perpendicular to the horizontal axis, the following procedure is adopted prior to making necessary adjustments.

(a) The theodolite is set up at T. After proper levelling, a ranging rod is fixed at A by looking through the telescope keeping the upper and lower clamps fixed.
(b) By transisting the telescope a ranging rod is fixed at B (Fig. 9.32(a)).
(c) The upper clamp is loosened and by turning the vernier plate the ranging rod at A is again bisected.
(d) If the ranging rod at B is seen bisected after transiting the telescope, the vertical hair is perfect.
(e) If not, the amount of error is noted. (In Fig. 9.32(b), let BB_1 be the total error.)

Adjustment

(a) A position is marked by a ranging rod at B′ where B_1B' is one-fourth of the total error (Fig. 9.32(b)).

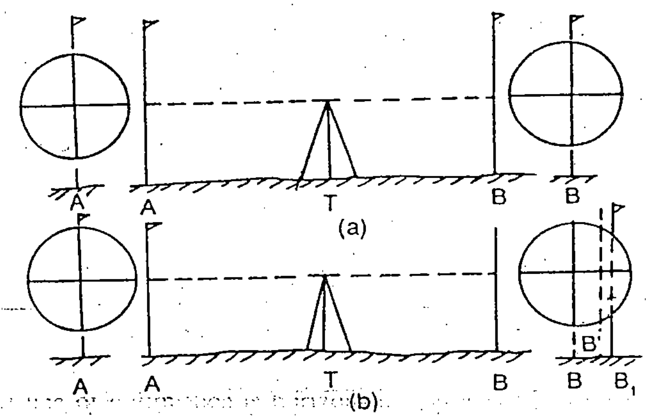


Fig. 9.32

(b) The vertical hair is shifted by turning the horizontal diaphragm screws, to bisect the ranging rod at B′.

(c) During adjustment, one-fourth of the total error is taken into consideration because the actual error is magnified four times as the telescope was turned twice in the vertical plane.

Third adjustment To make the horizontal axis perpendicular to the vertical axis, the following procedure is adopted before making necessary adjustments. (See Fig. 9.33):

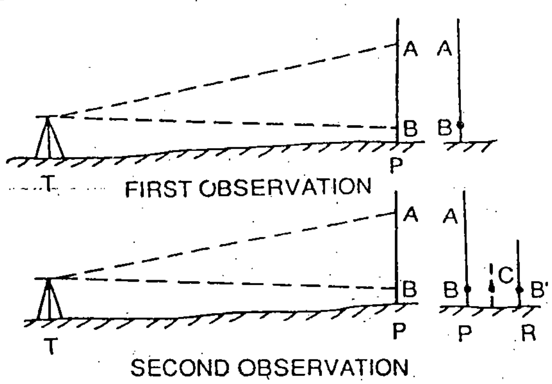


Fig. 9.33

1. The theodolite is set up at T some distance away from a pole (or wall) P.
2. The plate bubble is perfectly levelled. Looking through the telescope, a well-defined point A is marked on the pole. The upper and lower clamp screws are kept fixed.
3. The telescope is lowered and another point B is marked near the base of the pole in the same line of sight.
4. The upper clamp is loosened and the telescope is turned through 180°. By transiting it, the mark A is again bisected. The telescope is then lowered. If the line of sight bisects the mark B, then the adjustment is perfect.

5. If not, another point B′ is marked on a ranging rod R at the same level as B.

Adjustment

1. A point C is marked (in a suitable way) mid-way between B and B′.
2. The point C is bisected by the telescope and the upper clamp is tightened.
3. The telescope is now raised. This time the line of sight will not bisect A.
4. The adjustable end of the horizontal (trunnion) axis is raised or lowered until the line of sight bisects the mark A.
5. The procedure is repeated several times until the correction is perfect.

Fourth adjustment To make the axis of the telescope level (altitude bubble) parallel to the line of collimation, the procedure of adjustment is exactly similar to the "two-peg method" which is described in detail in Sec. 5.20.

Fifth adjustment This adjustment is made in order to ensure that the vertical circle reads zero when the line of collimation is horizontal.

This adjustment is not required for the transit theodolite. This is because in such a theodolite the vernier is adjustable and clamped at zero (in the horizontal position) when the altitude bubble is centred. Again the zero of the vertical circle which is fixed to the telescope) can also be set in the horizontal position by turning the tangent screw.

In theodolites provided with non-adjustable verniers, the reading of the vernier may not be zero when the altitude bubble is centred. In such a case, the amount of angular error, known as "index error", is noted. The sign of the index error should be taken into account. Necessary correction has to be applied to the observed vertical angle according to the sign of index error.

9.24 TRIGONOMETRICAL LEVELLING TO FIND HEIGHTS OF OBJECTS

Case I—When the base of the object is accessible Suppose the height of a tower TP is to be determined. (See Fig. 9.34). The theodolite is placed at a suitable position O at a distance *D* from the tower.

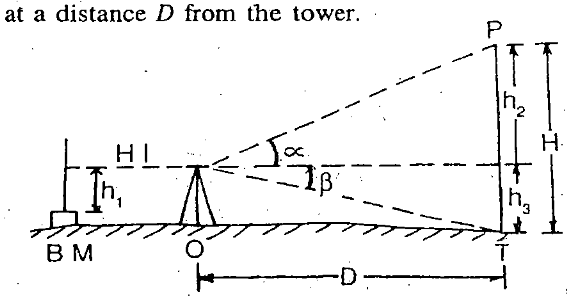


Fig. 9.34

Procedure

1. The theodolite is set up at O and perfectly levelled with respect to the plate bubble and the altitude bubble.
2. The distance *D* is measured by chain or tape or found out by the stadia method.

3. The angle of elevation α and angle of depression β are measured.
4. A BS reading is taken on the BM. Let it be h_1

Now, $h_2 = D \tan \alpha$
$h_3 = D \tan \beta$

and Total height $H = h_2 + h_3$
$= D \tan \alpha + D \tan \beta$ (1)

$\text{HI} = \text{BM} + h_1$
$\text{RL of P} = \text{BM} + h_1 + h_2$
$= \text{HI} + h_2$ (2)

Case II—when the base of the object is not accessible In this case, the angles of elevation are measured from two theodolite stations. For the sake of simplicity, these stations should be in the same vertical plane. But the elevations of the theodolites may not be equal.

1. When Height of Theodolite at A is Lower Than That at B Suppose the RL of the top of a hill P is to be determined (Fig. 9.35).

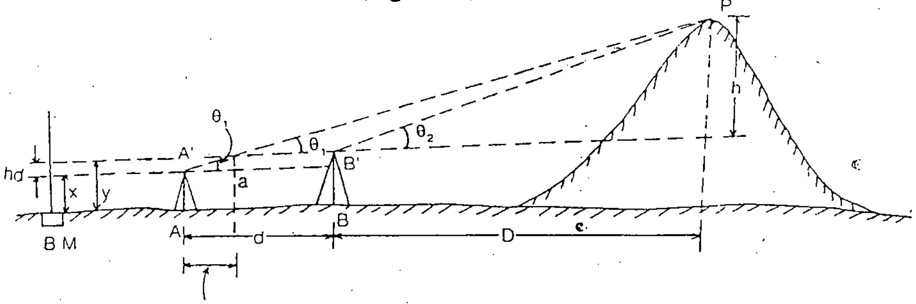


Fig. 9.35

Procedure

(a) The theodolite is set up at A and the angle of elevation θ_1 is measured. The BS reading is taken on the BM. Let it be x.

(b) The theodolite is shifted and set up at B at a distance d from A. The angle of elevation θ_2 is measured.

The BS reading is taken on the BM. Let it be y. Here, it should be remembered that A, B, and P are in the same vertical plane.

Now,

h_d = difference between BS readings = $y - x$
d = distance between instrument positions
D = distance between B and vertical line through P
h = height of hill above HI when instrument is at B
$A'a = h_d \cot \theta_1$

Again, $h = D \tan \theta_2$ (1)

and $h = \{D + (d - h_d \cot \theta_1)\} \tan \theta_1$ (2)

From Eqs (1) and (2),

$$D \tan \theta_2 = \{D + (d - h_d \cot \theta_1)\} \tan \theta_1$$

or $$D (\tan \theta_2 - \tan \theta_1) = (d - h_d \cot \theta_1) \tan \theta_1$$

or $$D = \frac{(d - h_d \cot \theta_1) \tan \theta_1}{\tan \theta_2 - \tan \theta_1}$$

From Eq. (1), $$h = \frac{(d - h_d \cot \theta_1) \tan \theta_1 \times \tan \theta_2}{\tan \theta_2 - \tan \theta_1}$$

$$\text{RL of P} = \text{BM} + Y + h$$

2. When Height of Theodolite at A is Higher Than That at B Follow the same procedure as before.

$$h_1 = \frac{(d + h_d \cot \theta_1) \tan \theta_1 \tan \theta_2}{\tan \theta_2 - \tan \theta_1}$$

$$\text{RL of P} = \text{BM} + y_1 + h_1$$

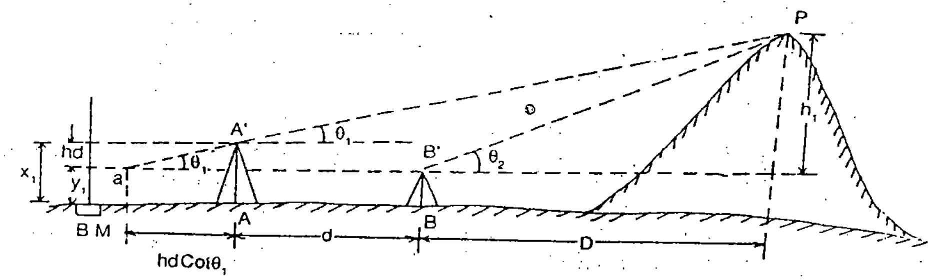


Fig. 9.36

SHORT QUESTIONS WITH ANSWERS FOR VIVA

Q. 1 What is a transit theodolite?
Ans. A transit theodolite is one in which the telescope can be revolved completely about the horizontal axis in a vertical plane.

Q. 2 What is a 12 cm theodolite?
Ans. A theodolite whose base circle (horizontal graduated circle) has a diameter of 12 cm is known as a 12 cm theodolite.

Q. 3 What are the functions of a theodolite?
Ans. The function of a theodolite are to measure the following quantities: (a) the horizontal angle, (b) the vertical angle, (c) the deflection angle, (d) the magnetic bearing, and (e) the horizontal distance.

Q. 4 Describe the location and function of the plate bubble and the altitude bubble?
Ans. The plate bubble is fixed over the horizontal graduated circle. It is levelled at the time of measuring the horizontal angle. The altitude bubble is fixed on top of the vertical vernier scale, and is levelled at the time of measuring the vertical angle.

Q. 5 What is the function of the shifting head?
Ans. Quick perfect centring may be done by moving the shifting head slowly.

Q. 6 State the procedure involved in bringing the bubble to the centre?

Ans. The bubble is first made parallel to any pair of foot screws. By turning the foot screws equally inwards or outwards the bubble is brought to the centre. Then the bubble is turned through 90°, and brought to the centre by means of the third foot screw. This process is repeated several times until the bubble is exactly in the centre for both directions of the bubble tube.

Q. 7 What are the functions of the clamp screw, tangent screw and clip screw?

Ans. Clamp screws are provided for fixing or releasing the main scale or vernier scale, and tangent screws for fine adjustment while bisecting objects. The clip screw is provided for levelling the altitude bubble.

Q. 8 What do the terms 'face left' and 'face right' mean?

Ans. When the vertical circle is on the left of the observer, the observation is said to be face left. When it is on the right of the observer, the observation is said to be face right.

Q. 9 What do the terms 'telescope normal' and 'telescope inverted' mean?

Ans. The face left position is known as telescope normal, and the face right position as telescope inverted.

Q. 10 What is an azimuth?

Ans. The true bearing of a line is also called its azimuth.

Q. 11 What is a trunnion axis?

Ans. The horizontal axis is also known as the trunnion axis.

Q. 12 What is transiting?

Ans. The process of turning the telescope in a vertical plane through 180° is known as transiting.

Q. 13 What does 'swinging the telescope' mean?

Ans. The process of turning the telescope in a horizontal plane is known as swinging. If the telescope is turned clockwise, there is said to be a right swing, and if it is turned anticlockwise, a left swing.

Q. 14 What is the least count of a theodolite?

Ans. The difference between the value of the smallest division of the main scale and that of the smallest division of the vernier scale known as the least count of the theodolite. It is the least value that can be measured by theodolite.

Q. 15 How can a theodolite be used as a level?

Ans. The altitude bubble is first perfectly levelled. Then the zero of the vertical circle (which is fixed to the telescope) is set at the zero of the vernier scale, and the telescope is clamped. Under this condition, the theodolite can be used as a level.

Q. 16 What is a deflection angle?

Ans. When a line (or alignment) changes its direction, the forward line makes an angle with the extension of the preceding line. This angle is known as the deflection angle.

Q. 17 Why are face left and face right observations taken?

Ans. These observations are taken to eliminate the error when the line of collimation is not perpendicular to the horizontal axis.

Q. 18 Why are two vernier readings taken?

Ans. Both vernier readings are taken to eliminate the error due to eccentricity of the inner and outer axes.

Q. 19. What do you know about repeating theodolites and direction theodolites?

Ans. A repeating theodolite consists of two vertical axes (one inner and one outer), two clamp screws and two tangent screws. The transit theodolite is a repeating theodolite.

A direction theodolite has one vertical axis, a single clamp screw and single tangent screw to control its rotation about vertical axis. Wild T-2 and T-3 theodolites fall under this category.

Q. 20 Name some modern theodolites.

Ans.
(a) The Watts microoptic theodolite
(b) The Wild T-2 theodolite
(c) The Tavistock theodolite
(d) The Wild T-3 precision theodolite
(e) The Wild T-4 universal theodolite.

Q. 21 What do the terms 'consecutive coordinates' and 'independent coordinates' mean?

Ans. The latitude and departure of a point with reference to the preceding line form what are called consecutive coordinates.

The coordinates of any point with respect to a common origin are known as independent coordinates.

Q. 22 What are latitude and departure?

Ans. The distance of a point from the origin along the north-south line is known as latitude and that along the east-west line is known as departure.

Q. 23 What are the sign conventions of latitudes and departures?

Ans.

NE quadrant— $\begin{cases} +L \\ +D \end{cases}$

SE quadrant— $\begin{cases} -L \\ +D \end{cases}$

SW quadrant— $\begin{cases} -L \\ -D \end{cases}$

NW quadrant— $\begin{cases} +L \\ -D \end{cases}$

Q. 24 What is Gale's table? What is the characteristic of this table?

Ans. The traverse table in which all information related to the traverse including the relevant independent coordinates, is tabulated, is known as Gale's table.

The characteristic of Gale's table is that the independent coordinates of all the traverse stations are brought to the NE quadrant by suitably assuming the coordinates of the starting stations of the traverse.

EXERCISES

1. What is the temporary adjustment of a theodolite? Describe the process of such adjustment.
2. Describe the process of measuring the horizontal angle.
3. Describe how you would measure vertical angles.
4. Describe the process of repetition and reiteration.
5. How would you measure deflection angles?
6. Describe the process of measuring the magnetic bearing of a line by theodolite.
7. How can a line be extended by a theodolite?
8. What are the methods of traversing by theodolite?
9. What are the methods of locating interior details in theodolite traversing?
10. Describe the methods of checking the accuracy of closed and open traverse.
11. What are the possible sources of error while using a theodolite? How can they be eliminated?
12. What is least count? How can it be found out for a particular instrument?

13. How is the closing error in a traverse balanced?
14. What are the fundamental lines of a theodolite? What should be the relation between them?
15. Describe the process of permanent adjustment of a transit theodolite.
16. How can the height of a tower be determined when it is inaccessible?
17. Define the following terms:
Centring, swinging, transiting, face left, face right, telescope normal, telescope inverted, temporary adjustment, permanent adjustment, and magnification.
18. ABCDA is a closed traverse in which the bearing of DA and length of BC have not been recorded. The rest of the field records are as follows:

Line	Length (m)	Bearing
AB	335	181°18′
BC	?	90°00′
CD	408	357°36′
DA	828	?

Find the missing data. (Ans. S 84°58′ W, 849.369 m)

19. The measured lengths and bearings of the sides of a closed traverse ABCDEA run in an anticlockwise direction, and are tabulated below:

Line	Length (m)	Bearing
AB	298.7	0°0′
BC	205.7	N 25°12′ W
CD	L_1	S 75°6′ W
DE	L_2	S 56°24′ E
EA	213.4	N 35°36′ E

Calculate the lengths of CD and DE.
(Ans. L_1 = 759.706 m, L_2 = 837.98 m)

20. The lengths and bearings of the sides of a closed traverse are represented below along with the latitudes and departures of known sides. Determine the bearing of AB and length of CD.

Line	Length (m)	Bearing	Latitude	Departure
AB	725.0	θ	—	—
BC	1,060.0	N 62°30′ E	+ 498.45	+ 940.24
CD	L	N 37°36′ E	—	—
DE	945.0	S 55°18′ W	− 537.99	− 776.92
EA	577.2	S 2°40′ W	− 576.63	− 26.85

(Ans. L = 947.00 m, θ = S 80°6′ W)

21. Choose the correct alternative for questions (i) to (xv).
(i) A theodolite in which the telescope can be revolved through a complete revolution in a vertical plane is known as a
(a) Non-transit theodolite (b) Tilting theodolite
(c) Transit theodolite
(ii) The size of the theodolite is defined according to the
(a) Diameter of graduated horizontal circle

(b) Length of the telescope
(c) Height of the standard
(iii) The face left position is also called
(a) Telescope inverted (b) Telescope normal
(c) Telescope reversed
(iv) If d is the smallest value of the main scale and v the smallest value of the vernier scale, then the least count of the vernier is given by
(a) $d - v$ (b) $v - d$ (c) $\frac{d}{v}$
(v) If f be the focal length of the objective and f_1 that of the eye-piece, then magnifying power is given by
(a) $\frac{f_1}{f}$ (b) $\frac{f}{f_1}$ (c) $f \times f_1$
(vi) If N be the number of sides of the traverse, then the sum of measured exterior angles should be equal to
(a) $(2n - 4) \times 90°$ (b) $(2n + 4) \times 90°$ (c) $(n + 4) \times 90°$
(vii) For important traverse surveys, the permissible angular error is
(a) $1'\sqrt{N}$ (b) $30''\sqrt{N}$ (c) $15''\sqrt{N}$
(viii) If θ be the RB of a line of length L, then departure is given by
(a) $L \cos \theta$ (b) $L \sin \theta$ (c) $L \operatorname{cosec} \theta$
(ix) In a closed traverse, the algebraic sum of departure and latitude must be equal to
(a) 90° (b) 180° (c) 0°
(x) Lines AB and BC intersect at B and ∠B is measured by theodolite; then the deflection angle of line BC is given by
(a) 180° − ∠B (b) 270° − ∠B (c) 90° + ∠B
(xi) Fine adjustment in a theodolite is done by the
(a) Focussing screw (b) Tangent screw (c) Clamp screw
(xii) The included angles of the traverse are meaured
(a) Clockwise (b) Anticlockwise (c) Either way
(xiii) For improved accuracy the included angle is measured by the
(a) Reiteration method (b) Repetition method
(c) Deflection angle method
(xiv) The characteristic of Gale's table is that the independent coordinates of all points are brought to the
(a) Fourth quadrant (b) First quadrant (c) Third quadrant
(xv) Balancing of traverse is done according to the
(a) Transit rule (b) Prismoidal rule (c) Trapezoidal rule

ANSWERS

Q. 21					
	(i) c	(ii) a	(iii) b	(iv) a	(v) b
	(vi) b	(vii) c	(viii) b	(ix) c	(x) a
	(xi) b	(xii) a	(xiii) b	(xiv) b	(xv) a

10

Curves

10.1 INTRODUCTION

During the survey of the alignment of a project involving roads or railways, the direction of the line may change due to some unavoidable circumstances. The angle of the change in direction is known as the deflection angle. For it to be possible for a vehicle to run easily along the road or railway track, the two straight lines (the original line and the deflected line) are connected by an arc (Fig. 10.1) which is known as the curve of the road or track.

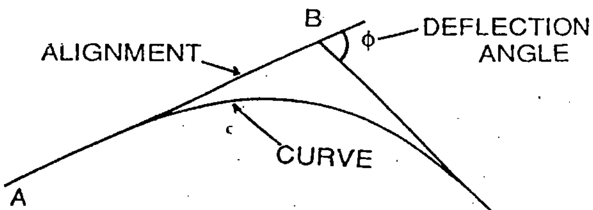


Fig. 10.1

When the curve is provided in the horizontal plane, it is known as a horizontal curve.

Again, along the alignment of any project the nature of the ground may not be uniform and may consist of different gradients (for instance, rising gradient may be followed by falling gradient and vice versa). In such a case, a parabolic curved path is provided in the vertical plane in order to connect the gradients for easy movement of the vehicles.

This curve is known as a vertical curve. The following are the different forms of curves:

- Curve
  - Horizontal Curve
    - Simple Curve
    - Compound Curve
    - Reverse Curve
    - Transition Curve
    - Lemniscate Curve
  - Vertical Curve

10.2 DEFINITIONS AND EXPLANATIONS OF DIFFERENT TERMS

1. Degree of curve The angle a unit chord of length 30 m subtends at the centre of the circle formed by the curve is known as the degree of the curve. It is designated as D (Fig. 10.2).

A curve may be designated according to either the radius or the degree of the curve.

When the unit chord subtends an angle of 1°, it is called a one-degree curve, when the angle is 2°, a two-degree curve, and so on.

It may be calculated that the radius of a one-degree curve is 1,719 m.

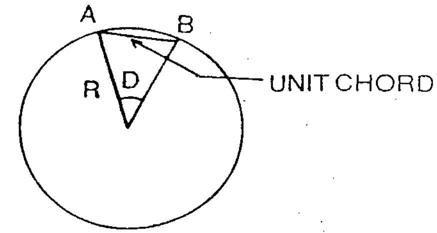


Fig. 10.2

2. Relation between radius and degree of curve Let AB be the unit chord of 30 m, O the centre, R the radius and D the degree of the curve (Fig. 10.3).

Here OA = R

AB = 30 m AC = 15 m

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

From triangle OAC,

$$\sin\frac{D}{2} = \frac{AC}{OA} = \frac{15}{R}$$

$$R = \frac{15}{\sin D/2}$$

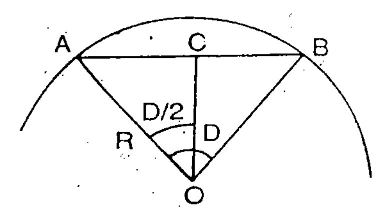


Fig. 10.3

When D is very small, sin $D/2$ may be taken as $D/2$ radians.

$$R = \frac{15}{(D/2) \times (\pi/180)} = \frac{15 \times 360}{\pi D} = \frac{1,718.9}{D} = \frac{1,719}{D} \quad \text{(approx)}$$

3. Superelevations When a particle moves in a circular path, then a force (known as centrifugal force) acts upon it, and tends to push it away from the centre.

Similarly, when a vehicle suddenly moves from a straight to a curved path, the centrifugal force tends to push the vehicle away from the road or track. This is because there is no component force to counterbalance this centrifugal force.

To counterbalance the centrifugal force, the outer edge of the road or rail is raised to some height (with respect to the inner edge), so that the sine component of the weight of the vehicle (W sin θ) may counterbalance the overturning force. The height through which the outer edge of the road or rail is raised is known as superelevation or cant.

In Fig. 10.4, P is the centrifugal force, W sin θ is the component of the weight of the vehicle, and h is the superelevation given to the road or rail.

For equilibrium,

$$W \sin\theta = \frac{WV^2}{gR}$$

or $W \times \frac{h}{b} = \frac{WV^2}{gR}$ (when θ is very small, $\sin\theta = \tan\theta = h/b$)

or $h = \frac{bV^2}{gR}$ for roads (1)

or $h = \frac{GV^2}{gR}$ for railways (2)

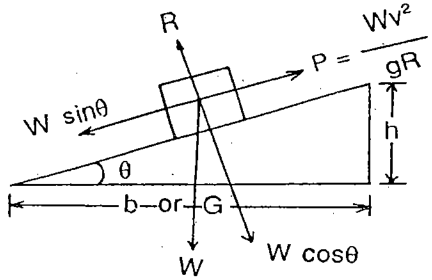


Fig. 10.4

where b = width of the road in metres
G = distance between centres of rails (gauge) in metres
R = radius of the curve in metres
g = acceleration due to gravity = 9.8 m/s^2
V = speed of the vehicle in metres per second
h = superelevation in metres.

4. Centrifugal ratio The ratio between the centrifugal force and the weight of the vehicle is known as centrifugal ratio.

$$\text{Centrifugal ratio (CR)} = \frac{P}{W} = \frac{WV^2}{gR \times W} = \frac{V^2}{gR}$$

$$\text{Allowable value for CR in roads} = \frac{1}{4}$$

$$\text{Allowable value for CR in railways} = \frac{1}{8}$$

10.2 TYPES OF HORIZONTAL CURVES

The following are the different types of horizontal curves:

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

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

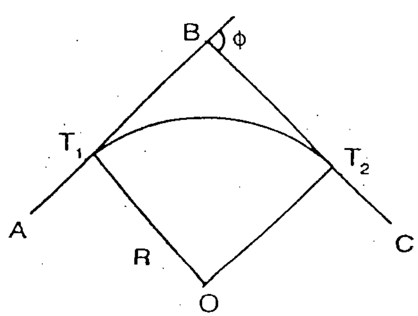


Fig. 10.5

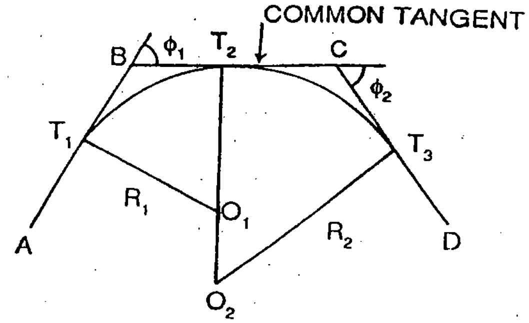


Fig. 10.6

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

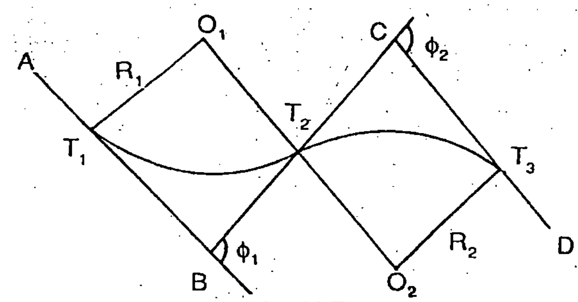


Fig. 10.7

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

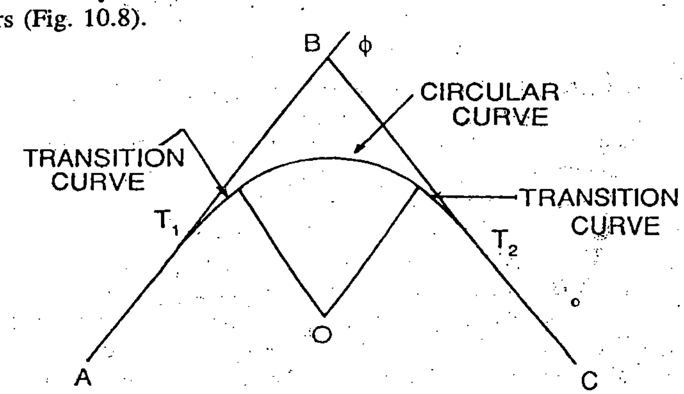


Fig. 10.8

5. Lemniscate curve A lemniscate curve is similar to a transition curve, and is generally adopted in city roads where the deflection angle is large. In Fig. 10.9, OPD shows the shape of such a curve. The curve is designed by taking a major axis OD, minor axis PP′, with origin O, and axes OA and OB. OP(ρ) is known as the polar ray, and α as the polar angle.

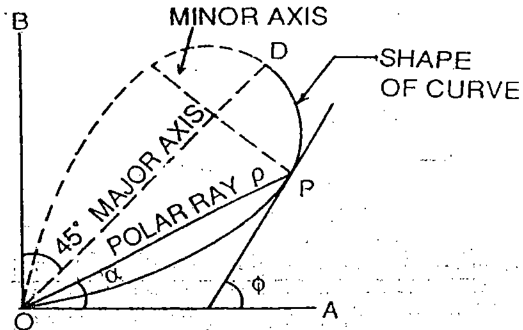


Fig. 10.9

Considering the properties of polar coordinates, the polar equation of the curve is given by

$$r = \frac{\rho}{3\sin 2\alpha}$$

where ρ = polar ray of any point
r = radius of curvature at that point
α = polar deflection angle

At the origin, the radius of curvature is infinity. It then gradually decreases and becomes minimum at the apex D.

$$\text{Length of curve OPD} = 1.3115\,K$$

where $K = 3r\sqrt{\sin 2\alpha}$

10.4 NOTATION USED WITH CIRCULAR CURVES

1. AB and BC are known as the tangents to the curve (Fig. 10.10).
2. B is known as the point of intersection or vertex.
3. The angle ϕ is known as the angle of deflection.
4. The angle I is called the angle of intersection.
5. Points T_1 and T_2 are known as tangent points.
6. Distances BT_1 and BT_2 are known as tangent lengths.
7. When the curve deflects to the right, it is called a right-hand curve, when it deflects to the left, it is said to be a left-hand curve.
8. AB is called the rear tangent and BC the forward tangent.
9. The straight line T_1DT_2 is known as the long chord.
10. The curved line T_1ET_2 is said to be the length of the curve.

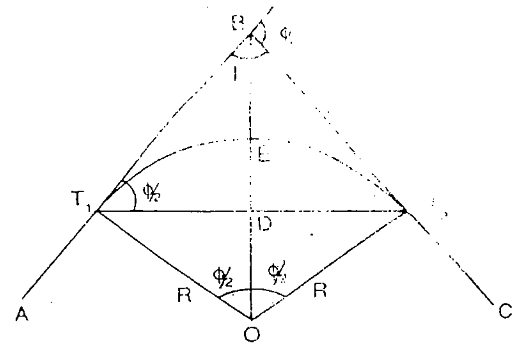


Fig. 10.10

11. The mid-point E of the curve T_1ET_2 is known as the apex or summit of the curve.
12. The distance BE is known as the apex distance or external distance.
13. The distance DE is called the versed sine of the curve.
14. R is the radius of the curve.
15. $\angle T_1OT_2$ is equal to the deflection angle ϕ
16. The point T_1 is known as the beginning of the curve or the point of curve.
17. The end of the curve (T_2) is known as the point of tangency.

10.5 PROPERTIES OF SIMPLE CIRCULAR CURVE

Consider Fig. 10.10.

1. If the angle of intersection is given, then

$$\phi = 180° - I \qquad (I = \text{angle of intersection})$$

2. If radius is not given, then

$$R = \frac{1{,}719}{D} \qquad (D = \text{degree of curve})$$

3. Tangent length BT_1 or BT_2 = $R \tan \phi/2$
4. Length of curve = length of arc T_1ET_2

$$= R \times \phi \text{ radians}$$

$$= \frac{\pi R\phi°}{180°}\text{ m}$$

Again, length of curve = $\frac{30\phi}{D}$ (if degree of curve D is given)

5. Length of long chord = $2T_1D$ = $2OT_1 \sin \phi/2$ = $2R \sin \phi/2$ m
6. Apex distance = BE = OB − OE

$$= R\sec\phi/2 - R = R(\sec\phi/2 - 1)\text{ m}$$

7. Versed sine of curve = DE = OF – OD

$$= R - R\cos\phi/2 = R(1 - \cos\phi/2)\ \text{m}$$

8. Full chord (peg interval): Pegs are fixed at regular intervals along the curve. Each interval is said to equal the length of a full chord or unit chord. The curve is represented by a series of chords, instead of arcs. Thus, the length of the chord is practically equal to the length of the arc. In usual practice, the length of the unit chord should not be more than 1/20th of the radius of the curve.

   In railway curves, the unit chords (peg intervals) are generally taken between 20 and 30 m. In road curves, the unit chord should be 10 m or less.

   It should be remembered that the curve will be more accurate if short unit chords are taken.
9. Initial subchord: Sometimes the chainage of the first tangent point works out to be a very odd number. To make it a round number, a short chord is introduced at the beginning. This short chord is known as the initial subchord.
10. Final subchord: Sometimes it is found that after introducing a number of full chords, some distance still remains to be covered in order to reach the second tangent point. The short chord introduced for covering this distance is known as the final subchord.
11. Chainage of first tangent point

    = chainage of intersection point – tangent length
12. Chainage of second tangent point

    = chainage of first tangent point + curve length

10.6 HORIZONTAL CURVE SETTING BY CHAIN-AND-TAPE METHOD

The following are the general methods employed for setting out curves by chain and tape:

1. Taking offsets or ordinates from the long chord
2. Taking offsets from the chord produced
3. Successively bisecting the arcs
4. Taking offsets from the tangents

A. Offsets or Ordinates from Long Chord

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

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

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

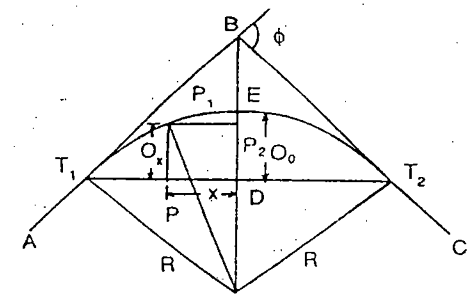


Fig. 10.11

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

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

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

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

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

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

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

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

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

   Thus, the mid-ordinate O_0 can be calculated from Eq. (1) or (2).
8. Considering the left half of the long chord, the ordinates O_1, O_2, ... are calculated at distances X_1, X_2, ... taken from D towards the tangent point T_1.

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

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

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

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

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

   or $$O_x = \sqrt{R^2 - x^2} - (R - O_0) \quad (3)$$
9. The ordinates for the right half are similar to these obtained for the left half.

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

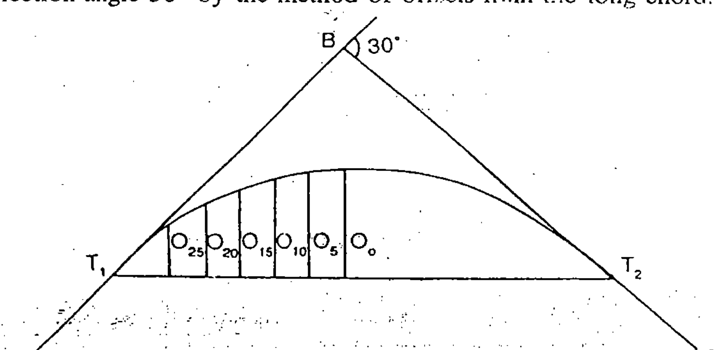


Fig. E.10.1

Solution

1. Tangent length $= R \tan \dfrac{\phi}{2}$
   $= 100 \times \tan 15° = 26.79$ m
2. Chainage of $T_1 = 150.50 - 26.79 = 123.71$ m
3. Curve length $= \dfrac{\pi R \phi°}{180°} = \dfrac{3.14 \times 100 \times 30°}{180°} = 52.36$ m
4. Chainage of $T_2 = 123.71 + 52.36 = 176.07$ m
5. Length of long chord $(L) = 2R \sin \phi/2$
   $= 2 \times 100 \times \sin 15° = 51.76$ m
6. The long chord is divided into two equal halves.
   Each half = 1/2 × 51.76 = 25.88 m
7. Mid-ordinate, $O_0 = R - \sqrt{R^2 - \left(\dfrac{L}{2}\right)^2}$
   $= 100 - \sqrt{100^2 - 25.88^2} = 3.41$ m
8. The ordinates are calculated at 5 m intervals starting from the centre towards T_1 for the left half.

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

$$O_{15} = \sqrt{(100^2 - 15^2)} - 96.59$$
$$= 98.87 - 96.59 = 2.28 \text{ m}$$
$$O_{20} = \sqrt{(100^2 - 20^2)} - 96.59$$
$$= 97.97 - 96.59 = 1.38 \text{ m}$$
$$O_{25} = \sqrt{(100^2 - 25^2)} - 96.59$$
$$= 96.82 - 96.59 = 0.23$$
$$O_{25.88} = \sqrt{(100^2 - 25.88^2)} - 96.59 = 0 \quad \text{(checked)}$$

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

Field procedure for measuring ordinates by long chord method

1. Let AB and BC be two tangents meeting at a point B, with deflection angle ϕ. (Refer to Fig. 10.12)

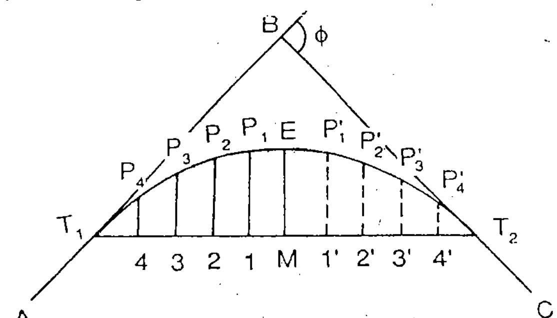


Fig. 10.12

2. The tangent length is calculated from the usual formula, and points T_1 and T_2 are marked on the ground with pegs.
3. The length of the long chord, T_1T_2, is calculated from the usual formula. The long chord is bisected at point M. The curve will be symmetrical on both sides of M.
4. The ordinates are calculated for the left half at some regular intervals. Points 1, 2, 3 and 4 are marked with pegs along the long chord as shown in Fig. 10.12.
5. Ordinates O_1, O_2, O_3 and O_4 are calculated from the usual formula.
6. Perpendiculars are set out at points 1, 2, 3, and 4. The calculated ordinates O_1, O_2, O_3 and O_4 identified along these perpendiculars and points P_1, P_2, P_3 and P_4 are marked with pegs.
7. In the right half, points 1′, 2′, 3′ and 4′ are marked with pegs and the corresponding ordinates (obtained for the left half) are set out to mark the points P'_1, P'_2, P'_3 and P'_4.
8. All these points P_1, P_2, ... and P'_1, P'_2, ... are on the curve. These points are joined by rope or thread to show the shape of the curve along the alignment (centreline) of the project.

B. Offsets from Chord Produced

1. Let

AB = rear tangent $\quad T_1$ = first tangent point

P_1 = first point on curve

$T_1C = T_1P_1 = b_1$, first chord or initial sub-chord

$CP_1 = O_1$ = first offset

$\angle CT_1P_1 = \alpha$ in radian

$\therefore \quad \angle T_1OP_1 = 2\alpha$ in radian

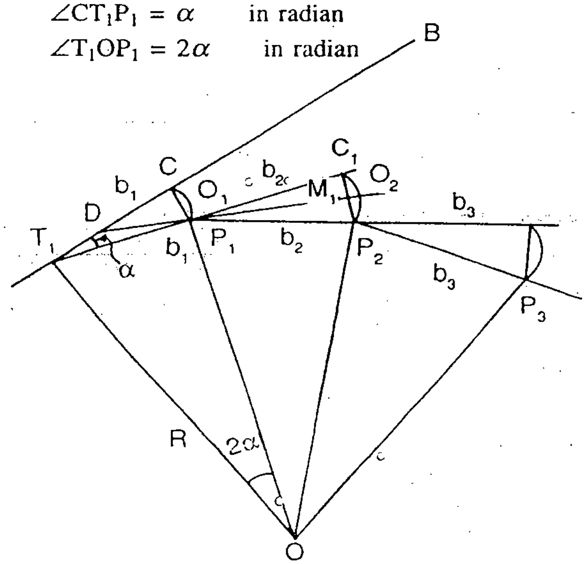


Fig. 10.13

Now assuming that

$$\text{Chord } T_1P_1 = \text{arc } T_1P_1 = R \times 2\alpha$$

$$\alpha = \frac{T_1P_1}{2R}$$

Again, $\quad$ Chord $CP_1 \sim$ arc CP_1

So

$$\text{First offset } O_1 = CP_1 = T_1P_1 \times \alpha$$

$$\therefore \quad O_1 = \frac{T_1P_1^2}{2R} = \frac{b_1^2}{2R} \; (T_1P_1 = b_1) \tag{1}$$

2. Again, let P_2 be the next point on the curve. The line T_1P_1 is extended and P_1C_1 is taken as the full chord b_2.

Now $\quad P_1C_1 = P_1P_2 \sim b_2$

and $\quad$ Chord C_1P_2 = arc $C_1P_2 = O_2$ (second offset)

At P_1, a tangent is drawn which meets the rear tangent AB and D and chord C_1P_2 at M_1.

Here, $\quad \angle C_1P_1M_1 = \angle DP_1T_1$ (opposite)

$\angle DP_1T_1 = \angle DT_1P_1$

So, $\quad \angle C_1P_1M_1 = \angle DT_1P_1 = \angle DT_1P_1 = \angle CT_1P_1$

Triangles CT_1P_1 and $C_1P_1M_1$ are similar.

$$\therefore \quad \frac{C_1M_1}{P_1C_1} = \frac{CP_1}{T_1P_1} \quad \text{or} \quad \frac{C_1M_1}{b_2} = \frac{O_1}{b_1}$$

$$\text{or} \quad C_1M_1 = \frac{b_2 \times O_1}{b_1} = \frac{b_2}{b_1} \times \frac{b_1^2}{2R} = \frac{b_1 b_2}{2R}$$

Here, M_1P_2 is the offset from the tangent at P_1.

So, according to Eq. (1),

$$M_1P_2 = \frac{(P_1P_2)^2}{2R} = \frac{b_2^2}{2R}$$

Second offset, $O_2 = C_1P_2 = C_1M_1 + M_1P_2$

$$\therefore \quad O_2 = \frac{b_1 b_2}{2R} + \frac{b_2^2}{2R} = \frac{b_2(b_1 + b_2)}{2R} \tag{2}$$

3. Similarly,

Third offset, $\quad O_3 = \dfrac{b_3(b_2 + b_3)}{2R} = \dfrac{b_3^2}{R} \quad$ (as $b_2 = b_3 = b_4 = \ldots$)

Fourth offset, $\quad O_4 = \dfrac{b_4(b_3 + b_4)}{2R} = \dfrac{b_4^2}{R} \quad$ and so on.

4. Last offset, $\quad O_n = \dfrac{b_n(b_{n-1} + b_n)}{2R}$

where $\quad b_n$ = final sub-chord $\quad b_{n-1}$ = last full chord

Example Two tangents intersect at a chainage of 1,000 m, the deflection angle being 30°. Calculate all the necessary data for setting out a circular curve of radius 200 m by the method of offsets from the chord produced, taking a peg interval of 20 m.

Solution Given data

$\phi = 30°$, $R = 200$ m, chainage of intersection point $= 1000$ m, and full chord $= 20$ m

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

$= 200 \times \tan 15° = 53.58$ m

2. Curve length $= \dfrac{\pi R \phi°}{180°} = \dfrac{\pi \times 200 \times 30}{180} = 104.72$ m

3. Chainage of first tangent point $= 1{,}000 - 53.58 = 946.42$ m

4. Chainage of second tangent point = 946.42 + 104.72
   = 1,051.14 m
5. Initial sub-chord = 950.00 − 946.42 = 3.58 m
6. No. of full chords of length 20 m = 5
   Chainage covered = 950.00 + 100.00 = 1,050.00 m
7. Final sub-chord = 1,051.14 − 1,050.00 = 1.14 m
8. First offset for initial sub-chord,

$$O_1 = \frac{b_1^2}{2R}$$

$$\therefore \quad O_1 = \frac{(3.58)^2}{2 \times 200} = 0.03 \text{ m}$$

Second offset for full chord,

$$O_2 = \frac{b_2(b_1 + b_2)}{2R} = \frac{20(3.58 + 20)}{2 \times 200} = 1.18 \text{ m}$$

Third offset for full chord, $O_3 = \frac{b_3^2}{R} = \frac{20^2}{200} = 2.0 \text{ m}$

Fourth offset for full chord, $O_4 = \frac{b_4^2}{R} = \frac{20^2}{200} = 2.0 \text{ m}$

Fifth offset for full chord, $O_5 = \frac{b_5^2}{R} = \frac{20^2}{200} = 2.0 \text{ m}$

Sixth offset for full chord, $O_6 = \frac{b_6^2}{R} = \frac{20^2}{200} = 2.0 \text{ m}$

Seventh offset for final sub-chord, $O_7 = \frac{1.14(20 + 1.14)}{2 \times 200} = 0.06 \text{ m}$

Note: There will be a total of seven offsets one for the initial sub-chord, five for full chords, and one for the final sub-chord. Here, the third through sixth offsets will be of the same length.

Field procedure for setting out curve by method of offsets from chord produced

1. Suppose AB and BC are the tangents, and B is the point of intersection (Fig. 10.14).
2. By calculating the tangent length, points T_1 and T_2 are marked on the ground with pegs.
3. The curve length is calculated, and then the chainages of T_1 and T_2 are found out.
4. The lengths of the initial and final sub-chords, and the number of full chords are determined.
5. The offsets for the initial sub-chord, full chord and final sub-chord are calculated.
6. The distance T_1C_1 is marked along the rear tangent AB so that T_1C_1 is equal to the initial sub-chord.

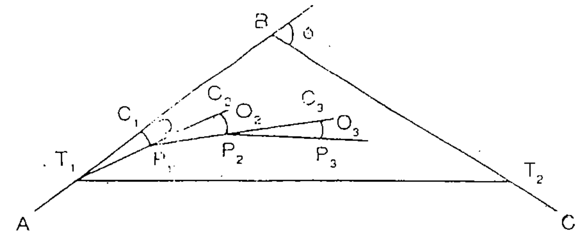


Fig. 10.14

7. The zero end of the tape is held at T_1, and an arc of radius T_1C_1 is drawn. From this arc, a distance C_1P_1 is cut off as the first offset (O_1).
8. The line T_1P_1 is now extended by a distance P_1C_2, which is the second chord (i.e. a full chord).
9. Then the zero end of the tape is held at P_1, and an arc of radius P_1C_2 is drawn. From this arc, a distance C_2P_2 is cut off as the second offset (O_2).
10. This process is continued until the second tangent point T_2 is reached.
11. The last point should concide with T_2. If it does not, the amount of error is found out. If the error is large, the entire operation should be repeated.

If the error is small, all the points are moved sideways by an amount proportional to the square of their distances from T_1. The error is thus distributed among all the points of the curve.

C. Successive Bisection of Arcs

1. In Fig. 10.15 AB and BC are two tangents intersecting at B, the deflection angle being ϕ. The tangent length is calculated, and tangent points T_1 and T_2 are marked on the ground with pegs.

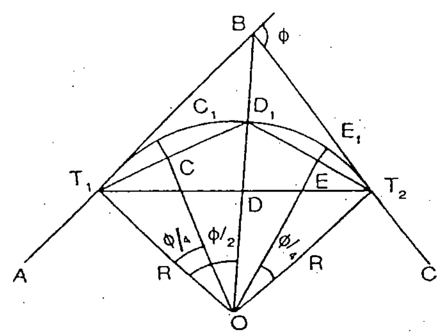


Fig. 10.15

2. T_1T_2 is the length of the long chord, which is bisected at D. A perpendicular is set out at this point and a distance DD_1 is cut off which is equal to the versed sine of the curve.

$$DD_1 = \text{versed sine of curve} = R\,(1 - \cos \phi/2)$$

3. Again the lengths T_1D_1 and T_2D_1 will serve as long chords for the curves between T_1 and D_1, and T_2 and D_1.
4. The distances T_1D_1 and T_2D_1 are measured and bisected at C and E. Now, the distances CC_1 and EE_1 will be equal to the versed sines of the small curve, which is given by

$$CC_1 = EE_1 = R\,(1 - \cos \phi/4)$$

The calculated distances CC_1 and EE_1 are cut off along the perpendiculars drawn at C and E.

5. This process is continued until the bisection of chords is not practically possible.

Then the points on the curve are joined by free hand.

D. Offsets from Tangents

Offsets from tangents may be:

1. Radial
2. Perpendicular

1. Radial Offsets

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

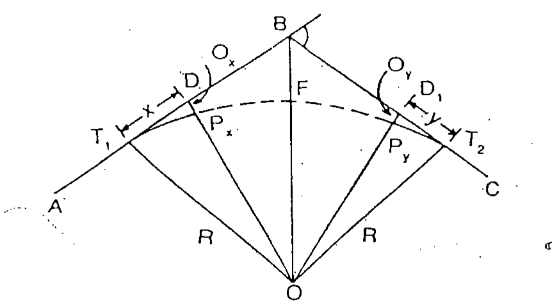


Fig. 10.16

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

$$T_1D = x$$

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

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

where

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

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

or

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

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

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

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

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

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

2. By perpendicular offsets

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

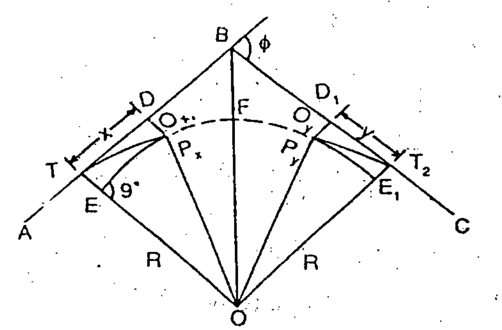


Fig. 10.17

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

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

From $\triangle OEP_x$, $\quad OP_x^2 = EP_x^2 + OE^2$

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

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

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

(c) This calculated distance O_x is set out along the perpendicular drawn at D to get the point P_x on the curve.
Similarly, by progressively increasing the value of x by a regular amount, a series of offsets are obtained. These are set out along the perpendicular drawn through the respective points.
Thus the left half of the curve is set out from T_1 up to the apex F.

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

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

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

10.7 INSTRUMENTAL METHOD—HORIZONTAL CURVE SETTING BY DEFLECTION ANGLE METHOD OR RANKINE'S METHOD

Let AB and BC be two tangents intersecting at B, the deflection angle being ϕ (Fig. 10.18). The tangent length is calculated and tangent points T_1 and T_2 are marked.

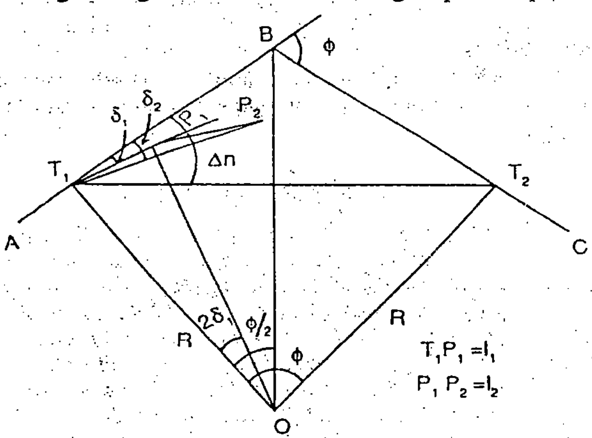


Fig. 10.18

Let
P_1 = first point on the curve,
T_1P_1 = l_1 length of first chord (initial sub-chord)
δ_1 = deflection angle for first chord
R = radius of the curve
Δ_n = total deflection for the chords

Here, $\angle T_1OP_1 = 2 \times \angle BT_1P_1 = 2\delta_1$

Again

$$\text{Chord } T_1P_1 \sim \text{arc } T_1P_1$$

Now,

$$\frac{\angle T_1OP_1}{l_1} = \frac{360°}{2\pi R}$$

$$2\delta_1 = \frac{360° \times l_1}{2\pi R}$$

or

$$\delta_1 = \frac{360° \times l_1}{2 \times 2\pi R} \text{ degrees}$$

or

$$= \frac{360 \times 60 \times l_1}{2 \times 2 \times \pi R} \text{ mins}$$

$$= \frac{1,718.9 \times l_1}{R} \text{ mins}$$

Similarly,

$$\delta_2 = \frac{1,718.9 \times l_2}{R} \text{ mins}$$

$$\delta_3 = \frac{1,718.9 \times l_3}{R} \text{ mins} \quad \text{and so on.}$$

Finally,

$$\delta_n = \frac{1,718.9 \times l_n}{R} \text{ mins}$$

Again, when degree of curve D is given,

$$\delta_1 = \frac{D \times l_1}{60} \text{ degrees}$$

$$\delta_2 = \frac{D \times l_2}{60} \text{ degrees} \quad \text{and so on.}$$

Finally,

$$\delta_n = \frac{D \times l_n}{60} \text{ degrees}$$

Arithmetical check: $\delta_1 + \delta_2 + \delta_3 + \ldots + \delta_n = \Delta_n = \phi/2$

Steps to remember for calculating data

1. Tangent length
2. Curve length
3. Chainage of first tangent point
4. Chainage of second tangent point
5. Initial sub-chord
6. Number of full chords
7. Final sub-chord
8. Deflection angle for initial sub-chord
9. Deflection angle for full chord
10. Deflection angle for final sub-chord
11. Arithmetical check
12. Data for field check
13. Setting out table

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

Solution Given data:

Radius = 250 m
Deflection angle ϕ = 180° – 150° = 30°
Chainage of intersection point = 1,250 m
Peg interval = 20 m
LC of vernier = 20″

1. Tangent length $= R \tan \phi/2$
$= 250 \times \tan 15° = 67.0$ m

2. Curve length $= \dfrac{\pi R \phi°}{180°} = \dfrac{\pi \times 250 \times 30°}{180°} = 130.89$ m

3. Chainage of first TP, T_1 = 1,250.0 – 67.0 = 1,183.0 m
4. Chainage of second TP, T_2 = 1,183.0 + 130.89 = 1,313.89 m
5. Length of initial sub-chord = 1,190.0 – 1,183.0 = 7.0 m
6. No. of full chords (20 m) = 6

Chainage covered = 1,190.0 + (6 × 20) = 1,310.00 m

7. Length of final sub-chord = 1,313.89 – 1,310.00 = 3.89 m
8. Deflection angle for initial sub-chord,

$$\delta_1 = \frac{1{,}718.9 \times 7.0}{250} \text{ mins} = 0°48'8''$$

9. Deflection angle for full chord,

$$\delta = \frac{1{,}718.9 \times 20}{250} \text{ mins} = 2°17'31''$$

10. Deflection angle for final sub-chord,

$$\delta_n = \frac{1{,}718.9 \times 3.89}{250} = 0°26'45''$$

11. Arithmetical check:

$$\text{Total deflection angle } (\Delta_n) = \delta_1 + 6 \times \delta + \delta_n$$

$$\phi/2 = \frac{30°}{2} = 15°$$

Here,

$$\Delta_n = 0°48'8'' + 6 \times 2°17'31'' + 0°26'45'' = 14°59'59''$$
$$= 15° \text{ (approximately)}$$

So, the calculated deflection angles are correct.

12. Data for field check:

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

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

13. Setting out table

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

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

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

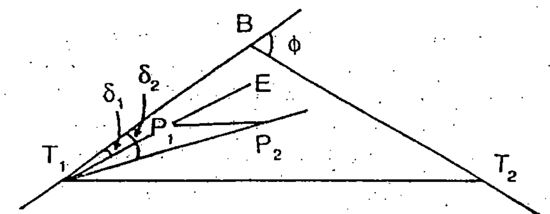


Fig. 10.19

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Field procedure of two-theodolite method This method is generally employed in railway curve setting, as it gives the correct location of points. In this method, no chain or tape is required to fix the points on the curve. It is mostly suitable when the ground surface is not favourable for chaining along the curve due to undulations. The two-theodolite method involves the following procedure:

1. All the necessary data for setting out the curve are calculated in the usual manner. The setting out table is also prepared.

2. Tangent points T_1 and T_2 are marked on the ground (Fig. 10.20).

3. A theodolite is centred over T_2 and levelled properly. Vernier A is set to 0° and the upper clamp is tightened. The lower clamp is released and by turning the telescope the ranging rod at T_1 is perfectly bisected. The lower clamp is now fixed.

4. The ranging rod at T_1 is taken off and another theodolite is centred over this

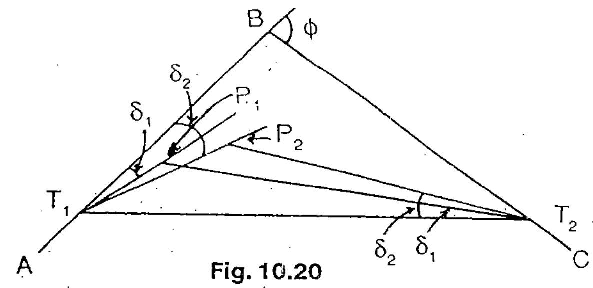


Fig. 10.20

point and levelled. Vernier A is set to 0°. The upper clamp is tightened. The lower clamp is released and the ranging rod at B is perfectly bisected. The lower clamp is now fixed.

5. The upper clamps of both theodolites are released. The first deflection angle (δ_1) is set on vernier A of both theodolites.

6. Then a point P_1 is so located that the lines of sight of both instruments intersect at it. So, P_1 is a point on the curve.

7. The next deflection angle (δ_2) is set on vernier A of both instruments. Again a point P_2 is so found that the lines of sight of both instruments intersect at it. So, P_2 is the next point on the curve.

8. This process is continued until all the deflection angles are set out, and all the points are marked.

9. Finally, when the total deflection angle (δ_n) is set out in both instruments, the line of sight of the theodolite at T_1 should bisect T_2 and that of the theodolite at T_2 should bisect B.

10.8 COMPOUND CURVE—CALCULATION OF DATA AND SETTING OUT

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

Notation

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

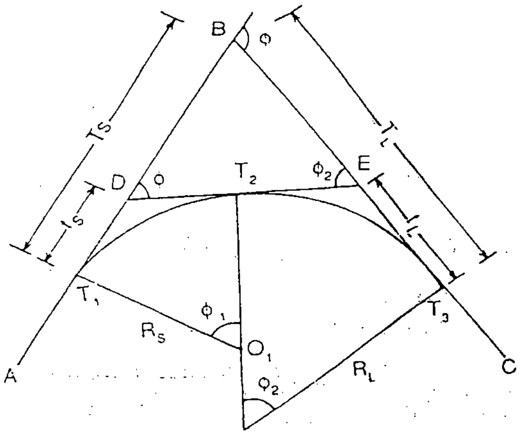


Fig. 10.21(a)

T_s = total tangent length of shortest side (BT_1)
T_L = total tangent length of longer side (BT_3)
t_s = tangent length of short curve
t_L = tangent length of long curve

Calculation of data

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

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

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

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

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

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

From ΔBDE,

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

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

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

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

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

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

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

where $\quad C_s$ = chord of short curve

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

where $\quad C_L$ = chord of long curve

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

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

Calculate all data necessary for setting out the compound curve.

Solution Consider Fig. 10.21(b).

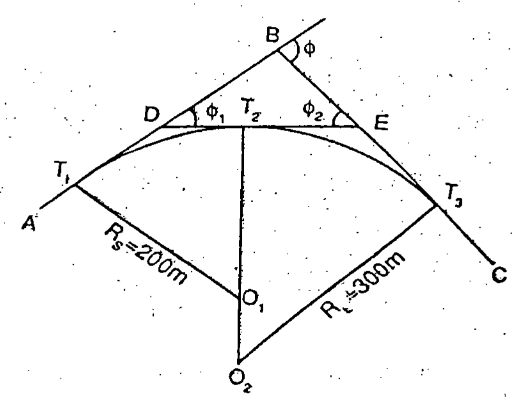


Fig. 10.21 (b)

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

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

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

3. $DE = DT_2 + T_2E = 53.58 + 109.19 = 162.77$ m

4. From ΔBDE,

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

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

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

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

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

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

6. Short curve length $= \dfrac{\pi \times 200 \times 30^\circ}{180^\circ} = 104.72$ m

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

8. Long curve length $= \dfrac{\pi \times 300 \times 40^\circ}{180^\circ} = 209.44$ m

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

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

Number of full chords = 5 (5 × 20 = 100 m)

Length of final sub-chord = 104.72 − 100 = 4.72 m

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

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

Check:

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

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

$$= 14^\circ 59' 59'' = 15^\circ \text{ (say)}$$

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

Number of full chords = 6 (6 × 30 = 180 m)

Length of final sub-chord = 209.44 − 180.00 = 29.44 m

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

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

Check:

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

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

$$= 19^\circ 59' 59'' = 20^\circ \text{ (say)}$$

Field procedure of setting out compound curve

1. In Fig. 10.22, AB is the rear tangent, BC the forward tangent and DE the common tangent.

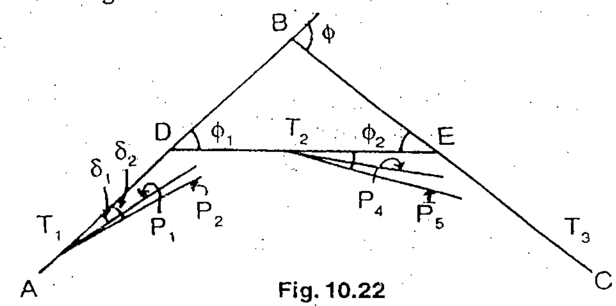


Fig. 10.22

2. Tangent lengths BT_1 and BT_3 are calculated and so are the curve lengths between T_1 and T_2, and T_2 and T_3.
3. The chainages of T_1, T_2, and T_3 are found out, and set out on the ground.
4. The deflection angles for the short and long curves are calculated, and the setting out table is prepared.
5. The theodolite is centred over T_1 and levelled properly. Then the deflection angles for the chords of the short curve are set out serially and points P_1, P_2, ... are marked until the tangent point T_2 is reached.
6. The theodolite is shifted and centred over T_2. Then the deflection angles for the chords of the long curve are set out serially and points P_4, P_5, ... are marked until the tangent point T_3 is reached.
7. All the points then joined to get the shape of the curve.
8. $\angle T_1T_2T_3$ is measured; it should be equal to

$$\left(180^\circ - \frac{\phi}{2}\right) \quad \text{or} \quad \left(180^\circ - \frac{\phi_1 + \phi_2}{2}\right)$$

10.9 REVERSE CURVE—CALCULATION OF DATA AND SETTING OUT

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

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

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

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

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

Notation

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

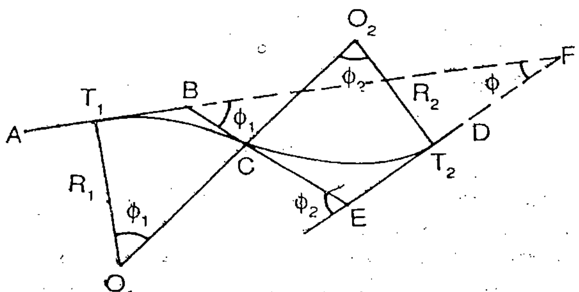


Fig. 10.23

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

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

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

Let ϕ_1 = angle of deflection for the first arc
ϕ_2 = angle of deflection for the second arc
ϕ = angle of intersection between AB and CD

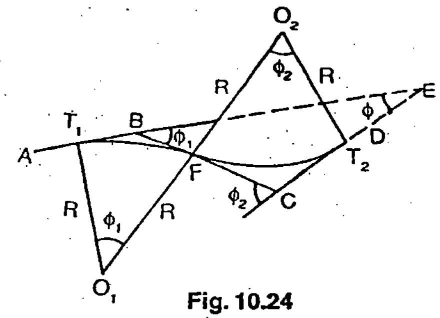


Fig. 10.24

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

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

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

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

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

Case II—When the straight lines are parallel In Fig. 10.25, PQ and RS are two parallel lines a distance y apart. It is required to connect the lines PQ and RS by a reverse curve having equal radii. Line AB is drawn parallel to PQ or RS through point C.

Let R = common radius
C = point of reverse curve
T_1 and T_2 = tangent points
ϕ = angle subtended at the centre by the curve

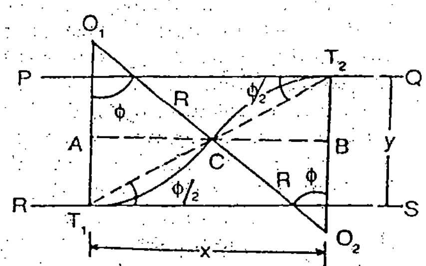


Fig. 10.25 (a)

$T_1T_2 = l$ = length of line joining T_1 and T_2
x = perpendicular distance between T_1 and T_2
y = perpendicular distance between lines PQ and RS.

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

1. Long chord for first curve, $T_1C = 2R \sin \phi/2$
2. Long chord for second curve, $T_2C = 2R \sin \phi/2$
3. Length $T_1T_2 = l$ $= 2R \sin \phi/2 + 2R \sin \phi/2$
$= 4R \sin \phi/2$

From ΔT_1T_2K $\quad \sin \phi/2 = \dfrac{y}{l}$

$$l = 4R \times \frac{y}{l} \quad \text{or} \quad l^2 = 4Ry$$

or $$l = \sqrt{4Ry}$$

4. $T_1A = O_1T_1 - O_1A = R - R\cos\phi = R(1 - \cos\phi)$
$T_2B = O_2T_2 - O_2B = R - R\cos\phi = R(1 - \cos\phi)$
$y = T_1A + T_2B = 2R(1 - \cos\phi)$
5. $x = AB = CA + CB = R\sin\phi + R\sin\phi = 2R\sin\phi$

Now the reverse curve may be set out by the method of offsets from the long chord, considering T_1C and T_2C as long chords. The procedure has been described in detail in subsection A of Sec. 10.6.

Example 1 While surveying along the alignment of a road, the magnetic bearings of the lines AB, BC and CD are measured as 80°, 110° and 60° respectively. The length of BC is 200 m, and the chainage of B is 950.00 m. Calculate the necessary data for setting out a reverse curve connecting the lines AB and CD by taking BC as the common tangent. The radii of both curves may be assumed equal.

Solution Consider Fig. 10.25(b).

$$\phi_1 = 110° - 80° = 30°$$
$$\phi_2 = \text{BB of BC} - (180° + 60°)$$
$$= 290° - 240° = 50°$$

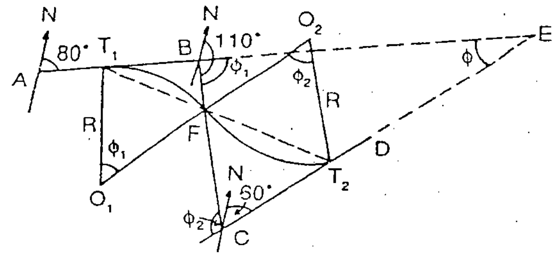


Fig. 10.25 (b)

Given data: BC = 200 m
Chainage of B = 950.00 m

Let F be the point of reverse curvature.

Now $\quad BC = BF + CF = R\tan\dfrac{\phi_1}{2} + R\tan\dfrac{\phi_2}{2}$

That is, $\quad 200 = R\tan 15° + R\tan 25° = 0.7342R$

$\therefore \quad R = 272.40$ m

1. Tangent length $T_1B = R\tan 15° = 272.40 \times 0.2679 = 72.97$ m
2. Chainage of T_1 $= 950.00 - 72.97 = 877.03$ m
3. First curve length $= \dfrac{\pi R\phi_1°}{180°} = \dfrac{\pi \times 272.40 \times 30°}{180°} = 142.62$ m
4. Chainage of F $= 877.03 + 142.62 = 1{,}019.65$ m
5. Second curve length $= \dfrac{\pi R\phi_2°}{180°} = \dfrac{\pi \times 272.40 \times 50}{180} = 237.71$ m
6. Chainage of T_2 $= 1{,}019.65 + 237.71 = 1{,}257.36$ m
7. Length of long chord for small curve,
$$T_1F = 2R\sin\frac{\phi_1}{2} = 2 \times 272.40 \times \sin 15° = 141.00 \text{ m}$$
8. Length of long chord for large curve,
$$T_2F = 2R\sin\frac{\phi_2}{2} = 2 \times 272.40 \times \sin 25° = 230.24 \text{ m}$$

Setting the first curve: The long chord T_1F is divided into two equal parts, each of length 70.5 m.
Offsets for the left half are calculated, taking peg intervals of 10 m.

$$\text{Mid-ordinate, } O_0 = R - \sqrt{R^2 - (L/2)^2} = 272.40 - \sqrt{(272.40)^2 - (70.5)^2}$$
$$= 9.28 \text{ m}$$

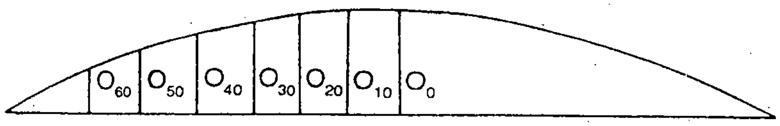


Fig. E. 10.2

Ordinate at distance 10 m,

$$O_{10} = \sqrt{(R^2 - x^2)} - (R - O_0)$$
$$= \sqrt{(272.40)^2 - (10)^2} - (272.40 - 9.28) = 9.09 \text{ m}$$
$$O_{20} = \sqrt{(272.40)^2 - (20)^2} - 263.12 = 8.54 \text{ m}$$
$$O_{30} = \sqrt{(272.40)^2 - (30)^2} - 263.12 = 7.62 \text{ m}$$
$$O_{40} = \sqrt{(272.40)^2 - (40)^2} - 263.12 = 6.32 \text{ m}$$
$$O_{50} = \sqrt{(272.40)^2 - (50)^2} - 263.12 = 4.65 \text{ m}$$
$$O_{60} = \sqrt{(272.40)^2 - (60)^2} - 263.12 = 2.58 \text{ m}$$

The offsets for the right half are similar to those for the left half.

Setting the second curve: The long chord T_2F is divided into left and right halves. The offsets for the left half are calculated exactly as described above. The offsets of the right half are similar to those of the left half. Then both the halves are set out in a similar manner.

Example 2 A reverse curve is to be set out to connect two parallel railway line 30 m apart. The distance between the tangent points is 150 m. Both the arcs have the same radius. The curve is to be set out by the method of ordinates from long chord, taking a peg interval of 10 m. Calculate the necessary data for setting the curve.

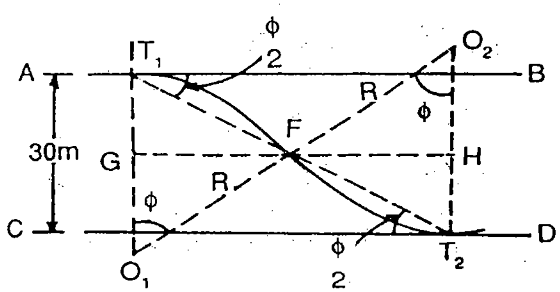


Fig. E. 10.3

Solution

AB and CD represent the parallel lines. The line T_1T_2 is the distance between the tangent points, which is given as 150 m.

We know that $T_1T_2 = T_1F + T_2F$

$$= 2R\sin\frac{\phi}{2} + 2R\sin\frac{\phi}{2} = 4R\sin\frac{\phi}{2}$$

or $$150 = 4R \times \frac{30}{150} \qquad \left(\sin\frac{\phi}{2} = \frac{30}{150}\right)$$

$$R = \frac{150 \times 150}{4 \times 30} = 187.5 \text{ m}$$

As $$\sin\frac{\phi}{2} = \frac{30}{150} = 0.2$$

$$\frac{\phi}{2} = 11°30'$$

$\therefore$ $$\phi = 23°0'$$

Horizontal distance between T_1 and T_2,

$$GH = GF + FH = R\sin\phi + R\sin\phi$$
$$= 2R\sin\phi = 2 \times 187.5 \times \sin 23° = 146.52 \text{ m}$$

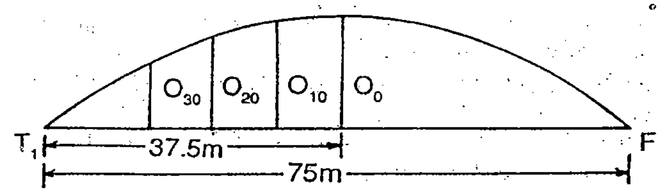


Fig. E. 10.4

The reverse curve is symmetrical, so the long chord for each curve = $T_1F = T_2F$ = 75 m. The long chord of the first curve is divided into two halves (left and right), and ordinates are calculated for the left half. The ordinates for the right half will be similar to those for the left half. Taking a peg interval of 10 m, the ordinates for the left half are calculated as follows:

Mid-ordinate, $$O_0 = R - \sqrt{R^2 - \left(\frac{L}{2}\right)^2} = 187.5 - \sqrt{(187.5)^2 - (37.5)^2} = 3.78 \text{ m}$$

$$O_{10} = \sqrt{R^2 - 10^2} - (R - O_0)$$
$$= \sqrt{(187.5)^2 - (10)^2} - (187.5 - 3.78) = 3.51 \text{ m}$$
$$O_{20} = \sqrt{(187.5)^2 - (20)^2} - 183.72 = 2.71 \text{ m}$$
$$O_{30} = \sqrt{(187.5)^2 - (30)^2} - 183.72 = 1.36 \text{ m}$$

Note: The ordinates for the second curve are similar to those for the first.

10.10 TRANSITION CURVE—CALCULATION AND SETTING OUT

1. Transition curve A curve of varying radius is known as a transition curve. The radius of such a curve varies from infinity to a certain fixed value. A transition curve is provided on both ends of a circular curve. The curvature varies from zero at the tangent point to a definite value just at the junction with the circular curve. Transition curves are provided in railway tracks to ensure safe running of trains.

Objectives of providing transition curves Transition curves are provided for the following reasons:

(a) To provide the superelevation gradually from zero at the tangent point to the specified amount on the circular curve.
(b) To maintain a constant proportion between superelevation and rate of change of curvature.
(c) To avoid overturning of the train.
(d) To minimise wear and tear of rail section due to unusual friction at point of turning.

2. Requirement of ideal transition curve The following conditions should be fulfilled while providing a transition curve:

(a) The specified amount of superelevation should be attained exactly at the junction with the circular curve.
(b) The rate of change of superelevation should be the same as that of the curvature.
(c) The radius of the transition curve should be equal to that of the circular curve exactly at the junction.
(d) It should meet the tangent point tangentially.
(e) It should meet the circular curve tangentially.

3. Combined or composite curve When transition curves are introduced at both ends of a circular curve, the resulting curve is known as a combined or composite curve.

4. Notation used with combined curves In Fig. 10.26:

(a) AB is the rear or back tangent.
(b) BC is the forward tangent.
(c) T_1 is known as the first tangent point, the starting point of the curve, or the point of curve.
(d) T_2 is called the second tangent point, finishing point of the curve, or point of tangency.
(e) The angle Δ is known as the deflection angle.
(f) The angle I is called the angle of intersection.
(g) The angle ϕ is known as the spiral angle.
(h) The distance T_1E or T_2D is the length of the transition curve.
(i) The arc EPD is the length of the circular curve.
(j) $T_1'E_1$ or $T_2'D_1$ is known as the shift of the curve.
(k) BT_1 or BT_2 is the total tangent length.

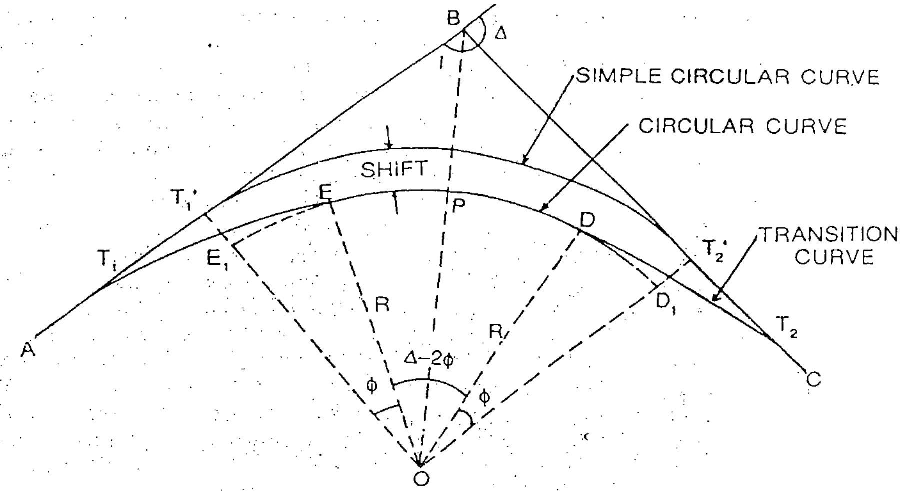


Fig. 10.26

(l) The length of arc T_1EPDT_2 is the length of the combined curve.
(m) The angle $(\Delta - 2\phi)$ is known as the central angle.

5. Condition of an ideal transition curve According to the requirement of an ideal transition curve, the superelevation should be increased uniformly with increase of centrifugal force, at a constant rate. The speed of the vehicle is assumed to be constant.

(a) So, the centrifugal force is proportional to the length of the transition curve, i.e.
$$P \propto L$$
or
$$\frac{WV^2}{gR} \propto L$$
where P = centrifugal force
L = length of transition curve

Here, W, V and g are constant.
$$\therefore \quad \frac{1}{R} \propto L \quad \text{or} \quad LR = \text{a constant}$$

(b) Again, the superelevation (h) is also proportional to the length of the transition curve (L), i.e.
$$h \propto L \propto \frac{WV^2}{gR}$$
Hence, we get
$$L \propto \frac{1}{R} \quad \text{or} \quad LR = \text{a constant}$$

Thus it is seen that the fundamental condition for a curve to be a transition curve is that the radius of curvature should be inversely proportional to the length.

Such a curve is also known as a clothoid or true spiral.

6. Intrinsic equation of ideal transition curve In Fig. 10.27:

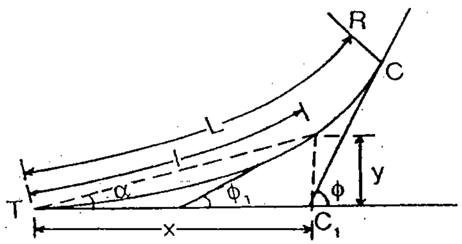


Fig. 10.27

TA = rear tangent
T = starting point of transition curve
C = point of junction between transition curve and circular curve
P = any point on transition curve
r = radius of curve at P
R = radius of circular curve
ϕ = spiral angle (angle between tangent TA and tangent at C)
ϕ_1 = angle between tangent at P and tangent TA
l = distance between point P and T
L = total length of transition curve
x = abscissa of point P
y = ordinate of point P
α = deflection angle at point P

The fundamental requirement of a spiral curve is that the radius of curvature at any point be inversely proportional to its length. So, considering the point P,

$$r \propto \frac{1}{l} \quad \text{or} \quad \frac{1}{r} \propto l$$

or

$$\frac{1}{r} = ml \tag{1}$$

where m = constant of proportionality.
Again, for any curve,

$$\frac{d\phi_1}{dl} = \frac{1}{r} \quad \text{or} \quad d\phi_1 = \frac{1}{r} \times dl$$

or

$$d\phi_1 = ml \times dl$$

Integrating

$$\phi_1 = \frac{ml^2}{2} + C$$

when $\quad l = 0 \quad$ and $\quad \phi_1 = 0$

Then $\quad C = 0$

So,

$$\phi_1 = \frac{ml^2}{2} \tag{2}$$

At point C, $\quad l = L, \quad r = R \quad$ and $\quad \phi_1 = \phi$

From Eq. (1),

$$\frac{1}{R} = mL \quad \text{or} \quad m = \frac{1}{RL}$$

From Eq. (2), $\quad \phi_1 = \frac{l^2}{2RL}$ radian

$$\therefore \quad \phi = \text{spiral angle} = \frac{L^2}{2RL} = \frac{L}{2R} \text{ radian} \quad (\text{as } l = L) \tag{3}$$

This is the intrinsic equation of the ideal transition curve.
The rectangular coordinates of P are given by the equation.

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

and the deflection angle at P by: $\quad \alpha = \frac{l^2}{2RL}$ radian

i.e.

$$\alpha = \frac{573 l^2}{RL} \text{ min}$$

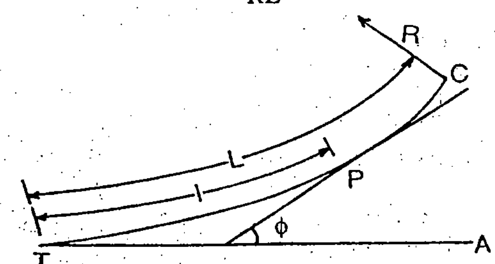


Fig. 10.28

7. Types of transition curves The following are the different types of transition curves.

(a) Euler's Spiral Such a spiral is shown in Fig. 10.28.

The equation of this curve, $\quad \phi = \frac{l^2}{2RL}$

This is the ideal equation of the transition curve,

where
P = a point on the curve
ϕ = angle made by tangent at P with initial tangent TA
l = distance between P and T
L = total length of transition curve
R = radius of circular curve

(b) Cubical Spiral Figure 10.29 shows a cubical spiral.
The equation of this curve is

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

where, y = perpendicular offset to any point from TA
l = distance of the point from T
L = total length of transition curve
R = radius of circular curve

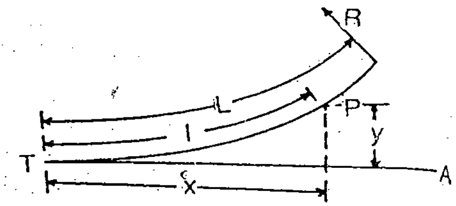


Fig. 10.29

(c) Cubic Parabola Figure 10.30 shows a cubic parabola.
The equation of this curve is

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

where
x = abscissa, i.e. horizontal distance between P and T
y = ordinate, i.e. perpendicular distance between P and TA

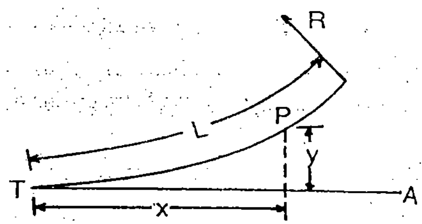


Fig. 10.30

Such curves are the most convenient for railway tracks.

(d) Lemniscate Curve Such a curve is shown in Fig. 10.31.
The equation of this curve is

$$r = \frac{\rho}{3 \sin 2\alpha}$$

where, r = radius of curvature at any point
ρ = polar ray at any point
α = polar deflection angle

Such curves are provided in roads.

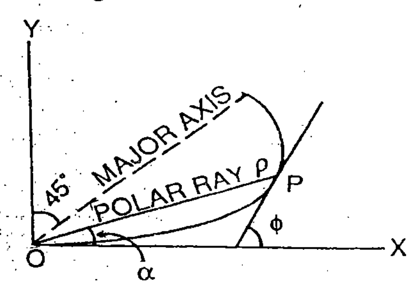


Fig. 10.31

8. Characteristics of transition curve

(a) Length The required length of a transition curve may be worked out in the following ways.

(i) *By adopting a definite rate of superelevation* The definite rate of superelevation is adopted as 1 in n, the value of n varying from 300 to 1,000.

$$\text{Length of transition curve,} \quad L = \frac{nh}{100} \text{ m}$$

where, h = amount of superelevation in centimetres.

Example Calculate the length of the transition curve required in order to attain a maximum superelevation of 15 cm, assuming the rate of superelevation to be 1 in 400.

$$\text{Length of transition curve,} \quad L = \frac{400 \times 15}{100} = 60 \text{ m}$$

(ii) *By considering arbitrary time rate of superelevation*
Length of transition curve,

$$L = \frac{h \times V}{x}$$

where, h = amount of superelevation
V = speed in metres per second
x = time rate in centimetres per second, it varies from 2.5 to 5 (cm/sec).

Example Calculate the length of transition curve required in order to attain a maximum superelevation of 15 cm, assuming a rate of superelevation of 3 cm/s. The speed of the vehicle is 50 km/h.

$$L = \frac{h \times V}{x}$$

$$V = \frac{50 \times 1{,}000}{60 \times 60} = \frac{500}{36} \text{ m/s}$$

$$h = 15 \text{ cm}$$

$$x = 3 \text{ cm/s}$$

$$\therefore \quad L = \frac{15 \times 500}{3 \times 36} = 69.44 \text{ m}$$

(iii) *By considering rate of change of radial acceleration*

$$\text{Radial acceleration on circular curve} = \frac{V^2}{R} \qquad (1)$$

$$\text{Time taken by vehicle to cover transition curve} = \frac{L}{V} \text{ s} \qquad (2)$$

Again, if K m/s^3 be the change of radial acceleration,

$$\text{Time taken to attain maximum radial acceleration} = \frac{V^2}{k \times R} \text{ s} \qquad (3)$$

From Eqs (2) and (3),

$$\frac{L}{V} = \frac{V^3}{k \times R}$$

$$L = \frac{V^3}{K \times R} \qquad (4)$$

Example Calculate the length of the transition curve, when rate of radial acceleration is 30 cm/s^3, allowable speed on curve is 60 km/h and the radius of the circular curve is 200 m.

Here,
$$V = \frac{60 \times 1{,}000}{60 \times 60} = 16.66 \text{ m/s}$$
$$K = 30 \text{ cm/s}^3 = 0.3 \text{ m/s}^3$$
$$R = 200 \text{ m}$$
$$L = \frac{V^3}{K \times R} = \frac{(16.66)^3}{0.3 \times 200} = 77 \text{ m}$$

b. Spiral Angle From the intrinsic equation of the ideal transition curve, we know that

$$\phi_1 = \frac{l^2}{2RL} \text{ radians}$$

when $\phi_1 = \phi$ and $l = L$

$$\phi = \frac{L^2}{2RL} = \frac{L}{2R} \text{ radians}$$

This angle ϕ is known as the spiral angle.

c. Deflection Angle In Fig. 10.32:

P = any point on transition curve
C = point of junction between transition curve and circular curve
ϕ = spiral angle
ϕ_1 = angle between tangent at P and initial tangent
α = deflection angle at P
x = abscissa of point P
y = ordinate of point P
R = radius of circular curve
l = length of transition curve between T and P

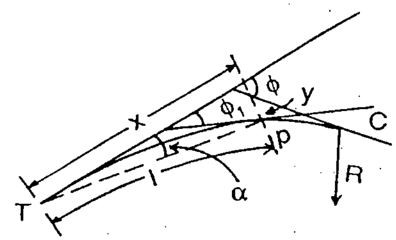


Fig. 10.32

Now,

$$\tan \alpha = \frac{y}{x} \quad (1)$$

From the equation of the cubical spiral, we know that

$$y = \frac{x^3}{6RL} = \frac{l^3}{6RL} \quad [x = l \text{ (approx)}] \quad (2)$$

Again, when α is small, $\tan \alpha = \alpha$.

From Eq. (1),

$$\alpha = \frac{1}{x} \times \frac{x^3}{6RL} = \frac{x^2}{6RL} = \frac{l^2}{6RL} \text{ radians} = \frac{573 l^2}{RL} \text{ mins}$$

(d) Shift In Fig. 10.33:

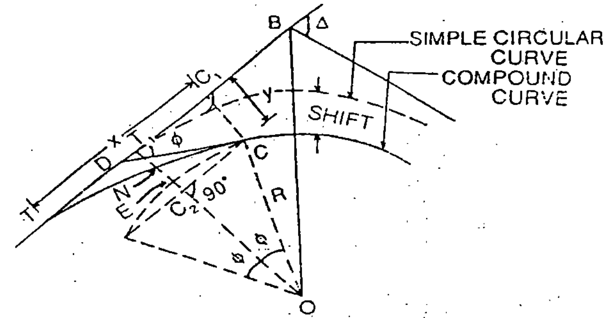


Fig. 10.33

TB = initial tangent
T = tangent point of combined curve
T_1 = tangent point simple circular curve
T_1E = S = shift
C = junction point of transition curve with circular curve
DC = common tangent
ϕ = spiral angle
N = intersection point of shift and transition curve
R = radius of circular curve
L = length of transition curve

(i) From the intrinsic equation of the ideal transition curve, we know that

$$\text{Spiral angle } \phi = \frac{L}{2R}$$

Now,
$$CE = R\phi = R \times \frac{L}{2R} = \frac{L}{2}$$

But CN is very nearly equal to CE.

$$\therefore \quad CN = \frac{L}{2}$$

Hence
$$TN = \frac{L}{2}$$

So, the shift T_1E bisects the transition curve at N.

(ii) Again, CC_2 is drawn perpendicular to OT_1, and CC_1 is drawn perpendicular to TB.

Now, $S = C_2T_1 - C_2E$ (C_2E = versed sine of curve with central angle 2ϕ)

$$= CC_1 - R(1 - \cos \phi) = y - R(1 - \cos \phi)$$

$$\therefore \quad S = y - R \times 2 \sin^2 \phi/2$$

But we know that

$$y = \frac{x^3}{6RL} = \frac{L^3}{6RL} = \frac{L^2}{6R} \qquad [x = L \text{ (approx)}]$$

When ϕ is small, $\sin \phi = \phi$.

$$\therefore \quad S = \frac{L^2}{6R} - R \times 2\left(\frac{\phi}{2}\right)^2 = \frac{L^2}{6R} - 2R \times \frac{\phi^2}{4}$$

$$= \frac{L^2}{6R} - 2R \times \frac{1}{4} \times \left(\frac{L}{2R}\right)^2 = \frac{L^2}{6R} - \frac{L^2}{8R} = \frac{L^2}{24R}$$

$$\therefore \quad \text{Shift} = \frac{L^2}{24R}$$

Again, $$T_1N = \frac{TN^3}{6RL} = \frac{(L/2)^3}{6RL} = \frac{L^2}{48R} \qquad \left(TT_1 = \overset{\circ}{T}N = \frac{L}{2}\right)$$

Now, $$\frac{\text{Shift}}{T_1N} = \frac{L^2}{24R} \times \frac{48R}{L^2} = 2$$

So, $T_1N = 1/2$ shift $= 1/2\ T_1E$

that means the transition curve bisects the shift.

9. Elements of combined curve The following points must be remembered while calculating the necessary data for setting out the combined curve (see Fig. 10.34).

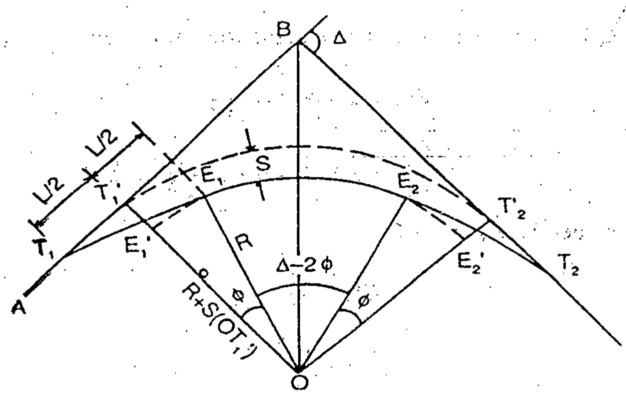


Fig. 10.34

(a) The length of the transition curve may either be given, or calculated from the formula

$$L = \frac{V^3}{KR}$$

(b) Shift of the curve, $$S = \frac{L^2}{24R}$$

where, R = radius of circular curve

(c) Tangent length of the combined curve (for cubic parabola),

$$BT_1 = BT_1' + T_1T_1' = OT_1' \tan\frac{\Delta}{2} + \frac{L}{2}$$

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

where, Δ = deflection angle

(d) Spiral angle,

$$\phi = \frac{L}{2R} \text{ radians} = \frac{L \times 180}{2R \times \pi} \text{ degrees}$$

(e) Central angle for circular curve $= (\Delta - 2\phi)$

(f) Length of circular curve $= \dfrac{\pi R(\Delta - 2\phi)}{180}$

(g) Length of combined curve $= T_1E_1 + E_1E_2 + E_2T_2$

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

(h) Chainage of point of commencement of curve,

$$T_1 = \text{chainage of } B - BT_1$$

(i) Chainage of E_1 = chainage of T_1 + length of transition curve

(j) Chainage of E_2 = chainage of E_1 + length of circular curve

(k) Chainage of point of tangency,

$$T_2 = \text{chainage of } E_2 + \text{length of transition curve}$$

(l) Setting out transition curve:

(i) According to deflection angle—The deflection angle at any point on the transition curve is equal to

$$\frac{573\, l^2}{RL} \text{ mins}$$

Check: Total deflection angle $= \frac{1}{3}\phi$

(ii) According to rectangular-coordinates—The coordinates of any point on the transition curve are given by the formula

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

(m) Setting out circular curve:

(i) By calculating deflection angle—The deflection angle at any point on circular curve is equal to

$$\frac{1{,}719 \times c}{R} \text{ mins}$$

Check: Total deflection angle = 1/2 × central angle

(ii) By taking offsets from long chord—This method is described in Sec. 10.6 subsection A.

(iii) By taking offsets from chord produced—This method is described in Sec. 10.6, subsection B.

(n) Field work:

(i) The first transition curve is set out from T_1 (first tangent point).

(ii) The circular curve is set out from E_1 (junction point of transition and circular curves).

(iii) The other transition curve is set out from T_2 (point of tangency).

Note: For a true spiral or cubic spiral,

$$\text{Tangent length of combined curve} = (R + S)\tan\frac{\Delta}{2} + \frac{L}{2}\left(1 - \frac{S}{5R}\right)$$

Example Two straights AB and BC intersect at chainage 1,000 m, the deflection angle being 40°. It is proposed to insert a circular curve of radius 300 m with a transition curve of length 90 m at each end. Calculate all data necessary for setting out the curve by the deflectionangle method, taking a peg interval of 20 m. Prepare the setting out table, taking the least count of theodolite as 20″.

Solution (a) Shift of curve $= \frac{L^2}{24R} = \frac{(90)^2}{24 \times 300} = 1.125$ m

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

$= (300 + 1.125)\tan 20° + 45 = 154.6$ m

(c) Spiral angle $(\phi) = \frac{L}{2R}$ radians

$= \frac{90}{2 \times 300} \times \frac{180}{\pi} = 8°36'$

(d) Central angle $= \Delta - 2\phi = 40° - 2 \times 8°36' = 22°48'$

(e) Length of circular curve $= \frac{\pi \times 300 \times 22°48'}{180°} = 119.38$ m

(f) Chainages:

Chainage of 1st TP = 1,000 − 154.6 = 845.4 m
Chainage of 1st junction point = 845.4 + 90 = 935.4 m
Chainage of 2nd junction point = 935.4 + 119.38 = 1,054.78 m
Chainage of 2nd TP (tangency) = 1,054.78 + 90 = 1,144.78 m

(g) Deflection angles for transition curve:

The chainage of the first point on the curve is taken as 850.0 m
Distance of 1st point from 1st TP = 850.0 − 845.4 = 4.6 m
Distance of 2nd point from 1st TP = 4.6 + 20 = 24.6 m
Distance of 3rd point from 1st TP = 24.6 + 20 = 44.6 m
Distance of 4th point from 1st TP = 44.6 + 20 = 64.6 m
Distance of 5th point from 1st TP = 64.6 + 20 = 84.6 m
Distance of 1st junction point = 90.00 m

Deflection angle for 1st point, $\alpha_1 = \frac{573 \times (4.6)^2}{300 \times 90} = 0°0'27''$

Deflection angle for 2nd point, $\alpha_2 = \frac{573 \times (24.6)^2}{300 \times 90} = 0°12'50''$

Deflection angle for 3rd point, $\alpha_3 = \frac{573 \times (44.6)^2}{300 \times 90} = 2°42'13''$

Deflection angle for 4th point, $\alpha_4 = \frac{573 \times (64.6)^2}{300 \times 90} = 1°28'34''$

Deflection angle for 5th point, $\alpha_5 = \frac{573 \times (84.6)^2}{300 \times 90} = 2°31'53''$

Deflection angle for junction, $\alpha_E = \frac{573 \times (90)^2}{300 \times 90} = 2°51'54''$

$= 2°52'0''$ (say)

Check: Total deflection angle $= \frac{1}{3}\phi = \frac{1}{3} \times 8°36' = 2°52'$ which is correct.

Table for setting out first transition curve

Point	Chainage (m)	Chord length to be measured	Deflection angle	Angle to be set	Remark
T_1	845.40	0	0	0	Starting point of curve
1st point	850.00	4.6	0°0′27″	0°0′20″	LC = 20″
2nd point	870.00	24.6	0°12′50″	0°13′0″	
3rd point	890.00	44.6	0°42′13″	0°42′20″	
4th point	910.00	64.6	1°28′34″	1°28′40″	
5th point	930.00	84.6	2°31′53″	2°32′0″	
1st Junction point	935.40	90	2°51′54″	2°52′0″	End of transition curve

(h) Deflection angle for circular curve:

Chainage of 1st junction = 935.40 m
The chainage of the first point on the circular curve is taken as 940.00 m.
Length of initial sub-chord = 940.00 − 935.40 = 4.60 m
No. of full chords of length 20 m = 5
Chainage covered = 940 + 100 = 1,040.00 m
Chainage of 2nd junction = 1,054.78 m
Length of final sub-chord = 1,054.78 − 1,040.00 = 14.78 m

Deflection angle for initial sub-chord $= \frac{1{,}719 \times 4.6}{300} = 0°26'21''$

$$\text{Deflection angle for full chord} = \frac{1{,}719 \times 20}{300} = 1°54'36''$$

$$\text{Deflection angle for final subchord} = \frac{1{,}719 \times 14.78}{300} = 1°24'41''$$

Check: Total deflection angle

$$= \frac{1}{2} \times \text{central angle} = \frac{1}{2} \times 22°48' = 11°24'$$

Here,

Total deflection angle

$$= 0°26'21'' + 5 \times 1°54'36'' + 1°24'41''$$
$$= 11°24'2'' = 11°24'0'' \quad \text{(say)}$$

Thus, the value of the total deflection angle has been arithmetically verified to be correct.

Setting out table for circular curve

Point	Chainage (m)	Chord length (m)	Deflection for each chord	Total deflection angle	Angles to be set	Remark
1st junction point	935.40	—	—	—	—	Starting point of circular curve
1st point	940.00	4.60	0°16′21″	0°26′21″	0°26′20″	LC = 20″
2nd point	960.00	20	1°54′36″	2°20′57″	2°21′0″	
3rd point	980.00	20	1°54′36″	4°15′33″	4°15′40″	
4th point	1,000.00	20	1°54′36″	6°10′9″	6°10′20″	
5th point	1,020.00	20	1°54′36″	8°4′45″	8°4′40″	
6th point	1,040.00	20	1°54′36″	9°59′21″	9°59′20″	
2nd junction point	1,054.78	14.78	1°24′41″	11°24′02″	11°24′0″	End of circular curve

(i) Deflection angles for second transition curve (with respect to second tangent point):

Chainage of tangency = 1,144.78 m

The chainage of the first point on the curve is taken as 1,140.00 m.

Distance of 1st point from tangency = 1,144.78 − 1,140.00 = 4.78 m

Distance of 2nd point from tangency = 4.78 + 20 = 24.78 m

Distance of 3rd point from tangency = 24.78 + 20 = 44.78 m

Distance of 4th point from tangency = 44.78 + 20 = 64.78 m

Distance of 5th point from tangency = 64.78 + 20 = 84.78 m

Distance of 2nd junction point from tangency = 90 m

$$\text{Deflection angle for 1st point} = \frac{573 \times (4.78)^2}{300 \times 90} = 0°0'29''$$

$$\text{Deflection angle for 2nd point} = \frac{573 \times (24.78)^2}{300 \times 90} = 0°13'2''$$

$$\text{Deflection angle for 3rd point} = \frac{573 \times (44.78)^2}{300 \times 90} = 0°42'33''$$

$$\text{Deflection angle for 4th point} = \frac{573 \times (64.78)^2}{300 \times 90} = 1°29'3''$$

$$\text{Deflection angle for 5th point} = \frac{573 \times (84.78)^2}{300 \times 90} = 2°32'32''$$

$$\text{Deflection angle for 2nd junction} = \frac{573 \times (90)^2}{300 \times 90} = 2°51'54''$$
$$= 2°52'0'' \quad \text{(say)}$$

Check: Total deflection angle $= \frac{1}{3}\phi = \frac{1}{3} \times 8°36' = 2°52'0''$

The value of the total deflection angle is thus arithmetically verified to be correct.

Setting out Table for second Transition Curve.

Point	Chainage (m)	Chord length to be measured (m)	Deflection angle	Angle to be set	Remark
T_2	1,144.78	—	—	—	Starting point of curve
1st point	1,140.00	4.78	0°0′29″	0°00′40″	LC = 20″
2nd point	1,120.00	24.78	0°13′2″	0°13′0″	
3rd point	1,100.00	44.78	0°42′33″	0°42′40″	
4th point	1,080.00	64.78	1°29′3″	1°29′0″	
5th point	1,060.00	84.78	2°32′32″	2°32′40″	
2nd junction	1,054.78	90	2°51′54″	2°52′0″	End of curve

10. Field procedure of setting out combined curve

(a) The tangent points T_1 and T_2 are marked with pegs by taking the tangent length along BA and BC. The points T_1' and C_1 are marked along AB so that

$$T_1T_1' = \frac{1}{2}L \quad \text{and} \quad T_1C_1 = \frac{2}{3}L$$

(see Fig. 10.35)

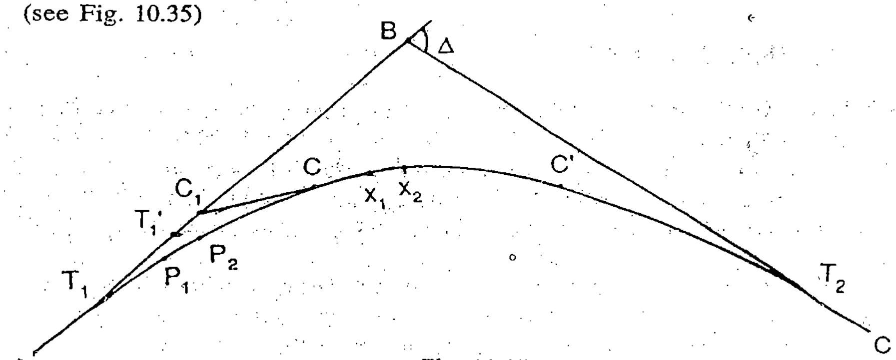


Fig. 10.35

(b) The theodolite is set up at T_1 and centred and levelled properly. Vernier A is set at zero and the upper clamp is fixed.

(c) The lower clamp is released and the ranging rod at B is bisected perfectly. The lower clamp is then tightened.

(d) The upper clamp is released and the first deflection angle α_1 is set off in vernier A. Then the distance of the first point is measured from T_1 so that the arrow at the first point P_1 is bisected by the line of sight of the theodolite also.

(e) Then the second deflection angle α_2 is set off and the distance of the second point is also measured from the tangent point T_1. In this manner, the point P_2 is fixed along the line of sight.

(f) The process is continued until the first junction point C is reached. It should be remembered that the distance of every point is measured from T_1. The total deflection angle should be (1/3) ϕ. Thus, the first transition curve is set out.

(g) To set the circular curve, the theodolite is shifted and set up at C. It is centred and levelled. Vernier A is set at $(2/3)\phi$ degrees behind zero (meaning at an angle of $360° - (2/3\ \phi)$. The upper clamp is then fixed.

(h) The lower clamp is released and the ranging rod at T_1 is bisected. After that, the lower clamp is tightened and the upper one released. The telescope is turned through the angle $(2/3)\phi$ so that vernier A reads zero. At this time, the line of sight should bisect the ranging rod at C_1. The upper clamp is tightened and the telescope is transited (i.e. turned through 180° in the vertical plane).

(i) The telescope is now pointed towards the direction of the common tangent. The upper clamp is released and the first deflection angle δ_1 set off on vernier A. The length of sub-chord is measured from C so that the arrow at the first point x_1 is bisected by the telescope.

(j) Then next deflection angle is set off and the distance of the next point is measured from x_1.

(k) The process is continued until the second junction point c′ is reached. Finally, the total deflection angle should be equal to half the central angle.

(l) Thus the central circular curve is set out. The theodolite is shifted and set up at T_2, and the second transition curve is set cut according to the procedure followed in the case of the first transition curve. All the necessary data are taken from the setting out table for the second transition curve. All distances are measured from T_2.

10.11 VERTICAL CURVES

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

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

2. Gradient The gradient is expressed in two ways:

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

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

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

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

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

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

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

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

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

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

Solution

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

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

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

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

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

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

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

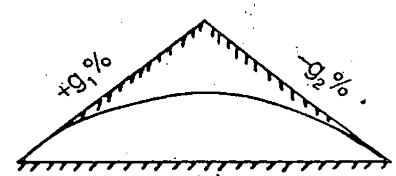


Fig. 10.36(a)

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

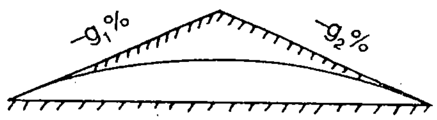


Fig. 10.36(b)

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

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

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

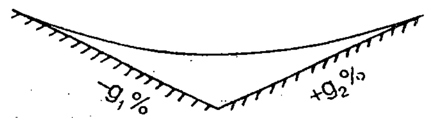


Fig. 10.36(c)

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

Fig. 10.36(d)

Then

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

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

The equation of the curve is

$$y = Cx^2$$

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

l = half-length of vertical curve

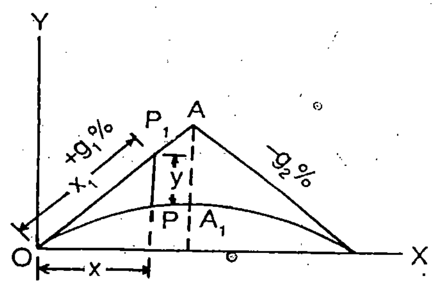


Fig. 10.37

Tangent correction at any point,

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

Thus,

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

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

where, $x_1, x_2 \ldots$ = distances taken along the slope measured from tangent point
l = half-length of vertical curve
g_1 and g_2 = percentages of grade

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

(a) The length of the vertical curve is assumed equal to the length of two tangents.
That is, $BT_1 + BT_2 = T_1B_1 + B_1T_2$
$= 2l$ (l = half length of vertical curve)

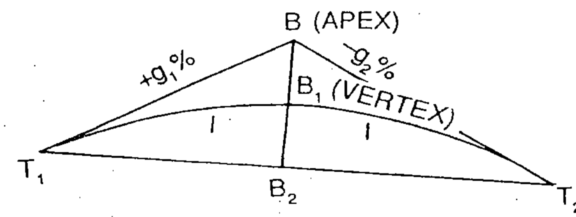


Fig. 10.38

(b) The curve is assumed to be equally long on either side of the apex point.
That is, $T_1B_1 = B_1T_2 = l$ so, $BT_1 = BT_2 = l$

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

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

(d) Chainage of T_1 = chainage of B – BT_1
(e) Chainage of T_2 = chainage of B + BT_2
(f) RL of T_1 = RL of B $\pm\, l \times \dfrac{g_1}{100}$
(g) RL of T_2 = RL of B $\pm\, l \times \dfrac{g_2}{100}$
(h) RL of B_2 = $\dfrac{1}{2}$ (RL of T_1 + RL of T_2)
(i) RL of B_1 = $\dfrac{1}{2}$ (RL of B + RL of B_2)
(j) Tangent correction at distance x,

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

(k) The tangent correction is deducted from the RL of a point on the grade to get the corresponding point on the curve.
(l) A setting out table is prepared.
(m) Since the curve is symmetrical, tangent corrections are calculated for one side of the point of intersection. The tangent corrections for the other side will be exactly the same.

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

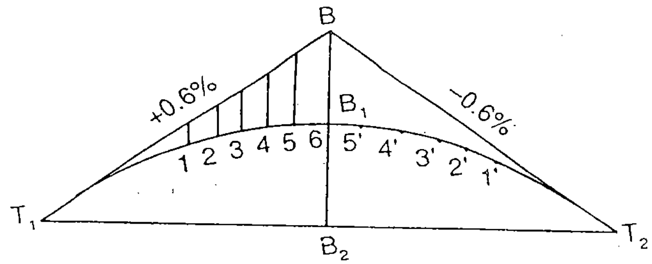


Fig. E. 10.5

Solution

(a) Length of vertical curve,

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

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

(b) Chainage of T_1 = 550 − 180 = 370 m

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

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

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

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

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

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

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

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

l being half the curve length.

Tangent correction at point 1,

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

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

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

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

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

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

(j) Reduced levels on grade:

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

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

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

(k) Reduced level on the curve:

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

Setting out table

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

10.12 PROBLEMS FACED IN CURVE SETTING

The process of curve setting may involve various problems, which have to be suitably tackled depending on the site condition. We shall now discuss a few of them.

The following are the different problems that occur:

1. The point of intersection may be inaccessible.
2. Both tangent points may be inaccessible.
3. It may not be possible to set out the full curve from one point.
4. There may be an obstacle across the curve.

1. Inacccesible point of intersection Consider Fig. 10.39. Let two straight lines AB and BC intersect at B, which is inaccessible. So, the deflection angle ϕ cannot be measured. Let us select two points D and E along AB and BC respectively. The distance DE is measured, and the angles θ_1 and θ_2 are measured by theodolite.

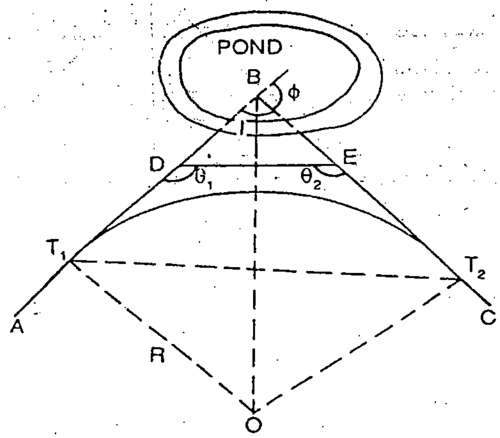


Fig. 10.39

Now,

$$\angle BDE = 180° - \theta_1$$

$$\angle BED = 180° - \theta_2$$

Angle of intersection, $I = 180° - (180 - \theta_1 + 180 - \theta_2)$

$$= (\theta_1 + \theta_2 - 180°)$$

Deflection angle, $\phi = 180° - (\theta_1 + \theta_2 - 180°)$

$$= \{360° - (\theta_1 + \theta_2)\}$$

Applying the sine rule in qBDE

$$\frac{BD}{\sin(180° - \theta_2)} = \frac{BE}{\sin(180° - \theta_1)} = \frac{DE}{\sin(\theta_1 + \theta_2 - 180°)}$$

$$BD = DE \times \frac{\sin(180° - \theta_2)}{\sin(\theta_1 + \theta_2 - 180°)}$$

$$BE = DE \times \frac{\sin(180° - \theta_1)}{\sin(\theta_1 + \theta_2 - 180°)}$$

Now, tangent length,

$$BT_1 = R \tan \frac{\{360° - (\theta_1 + \theta_2)\}}{2}$$

$$DT_1 = BT_1 - BD \quad \text{and} \quad ET_2 = BT_2 - BE$$

Now the tangent points are fixed by measuring distances DT_1 and ET_2. When T_1 and T_2 are fixed, the curve can be set out by any method.

2. Both tangent points being inaccessible Consider Fig. 10.40. In this case tangent points T_1 and T_2 are inaccessible, but intersection point B is accessible.

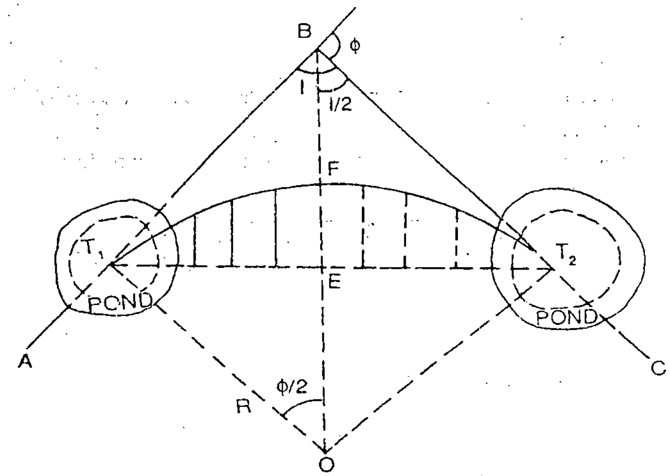


Fig. 10.40

(a) Tangent length $BT_1 = BT_2 = R \tan \phi/2$

(b) Curve length = $\dfrac{\pi R \phi°}{180°}$

(c) Length of long chord = $2R \sin \phi/2$

(d) Apex distance $BF = R(\sec \phi/2 - 1)$

(e) Versed sine $EF = R(1 - \cos \phi/2)$

All these data are calculated from the given formulae.

So,

Chainage of T_1 = chainage of B – BT_1

Chainage of T_2 = chainage of T_1 + curve length

The angle of intersection is bisected and along this line the apex distance and versed sine are set out to get the points F and E. At E a perpendicular to EF is drawn which represents the long chord. Now, the points on the curve are set out by the method of offsets from the long chord.

3. When full curve cannot be set out from one point The tangent points T_1 and T_2 are marked in the usual way. The theodolite is set up at T_1 and the points on the curve are set out as usual up to point P. Let the total deflection angle be Δ_p (see Fig. 10.41).

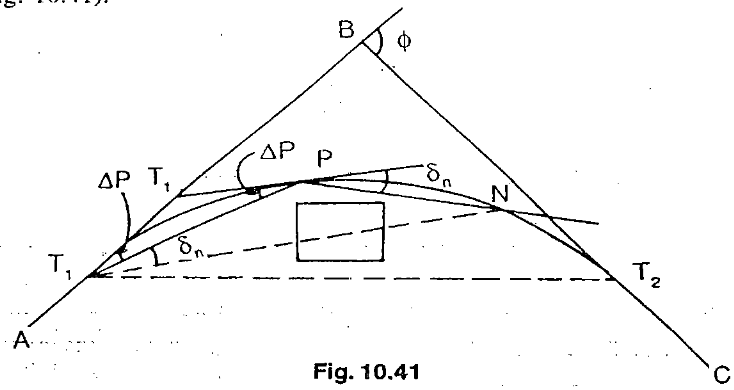


Fig. 10.41

The theodolite is shifted and set up at P. Vernier A is set at zero, and the ranging rod at T_1 is bisected. Then the angle Δ_P is set on vernier A, and a point T_1' is marked. The line $T_1'P$ is the tangent to the curve at P.

Vernier A is again set at zero and the ranging rod at T_1 is bisected. The telescope is now transited.

The deflection angle δ_n for the next point N is set and marked on the ground.

The process is repeated until all the points are located. The calculation of deflection angle and mode of setting out are the same as in Rankine's method.

4. Obstacle occurring across the curve Suppose a building comes across the curve (Fig. 10.42). From T_1, points P_1, ..., P_4 are marked. Then the total deflection angle for P_5 is set out. Let this angle be θ.

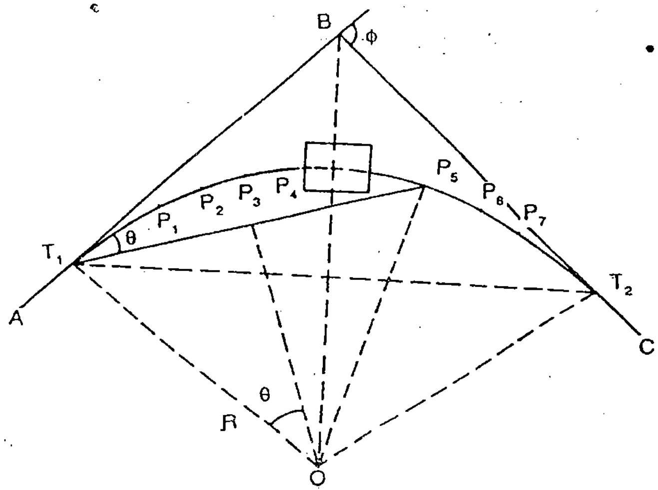


Fig. 10.42

Then the length of the long chord T_1P_5 is calculated as follows.

$$T_1P_5 = 2R \sin \theta$$

This calculated length is measured along the line T_1P_5 to locate the point P_5 on the curve. Then, normal procedure is followed in order to locate the remaining points on the curve.

10.13 WORKED OUT PROBLEMS ON HORIZONTAL CURVES

Problem 1 Two tangents intersect at a chainage of 1,320.5 m, the deflection being 24°. Calculate the following quantities for setting out a curve of radius 275 m.

(a) Tangent length
(b) Length of long chord
(c) Length of the curve
(d) Chainage of point of commencement and tangency
(e) Apex distance, and
(f) Versed sine of curve.

Solution

(a) Tangent length $= R \tan \phi/2 = 275 \times \tan 12° = 58.45$ m

(b) Length of long chord $= 2R \sin \phi/2$
$= 2 \times 275 \times \sin 12° = 114.35$ m

(c) Length of curve $= \dfrac{\pi R\phi}{180°} = \dfrac{\pi \times 275 \times 24°}{180°} = 115.19$ m

(d) Chainage of commencement = 1,320.50 − 58.45 = 1,262.05 m
Chainage of tangency = 1,262.05 + 115.19 = 1,377.24 m

(e) Apex distance $= R (\sec \phi/2 - 1)$
$= 275 (\sec 12° - 1) = 6.19$ m

(f) Versed sine of curve $= R (1 - \cos \phi/2)$
$= 275 (1 - \cos 12°) = 6.05$ m

Problem 2 Two straight lines AC and CB, to be connected by a 3° curve, intersect at a chainage of 2,760 m. The WCBs of AC and CB are 45°30′ and 75°30′ respectively. Calculate all necessary data for setting out the curve by the method of offsets from the long chord.

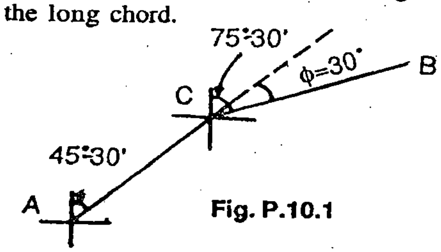


Fig. P.10.1

Solution

(a) Deflection angle $\phi = 75°30' - 45°30' = 30°$

(b) Radius of curve, $R = \frac{1,719}{3} = 573$ m

(c) Tangent length = $R \tan \phi/2 = 573 \times \tan 15° = 153.53$ m

(d) Curve length = $\frac{\pi R\phi}{180} = \frac{\pi \times 573 \times 30}{180} = 300.02$ m

(e) Length of long chord = $2R \sin \phi/2$
$= 2 \times 573 \times \sin 15° = 296.60$ m

(f) Chainage of point of commencement = 2,760 – 153.53 = 2,606.47 m

(g) Chainage of tangency = 2,606.47 + 300.02 = 2,906.49 m

(h) The long chord is divided into two equal halves. (i.e. the left half and the right half).

Each half = $\frac{1}{2} \times 296.60 = 148.3$ m

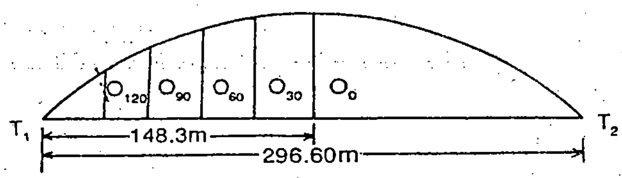


Fig. P.10.2

The ordinates for the left half are calculated. These are taken at 30 m intervals.

$$O_0 = R - \sqrt{R^2 - \left(\frac{L}{2}\right)^2}$$
$$= 573 - \sqrt{(573)^2 - (148.3)^2} = 573 - 553.47 = 19.52 \text{ m}$$

$$O_{30} = \sqrt{(573)^2 - (30)^2} - (573 - 19.52)$$
$$= 572.21 - 553.48 = 18.73 \text{ m}$$

$$O_{60} = \sqrt{(573)^2 - (60)^2} - 553.48 = 16.36 \text{ m}$$

$$O_{90} = \sqrt{(573)^2 - (90)^2} - 553.48$$
$$= 565.88 - 553.48 = 12.40 \text{ m}$$

$$O_{120} = \sqrt{(573)^2 - (120)^2} - 553.48$$
$$= 560.29 - 553.48 = 6.81 \text{ m}$$

$$O_{148.3} = \sqrt{(573)^2 - (148.3)^2} - 553.48$$
$$= 553.48 - 553.48 = 0$$

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

Problem 3 Two straight lines AB and BC intersect at a chainage of 510.23 m, the angle of intersection being 126°48′. The radius of the curve is 300 m. Calculate all data necessary for setting out the curve by the method of offsets from the chord produced. Assume a peg interval of 30 m.

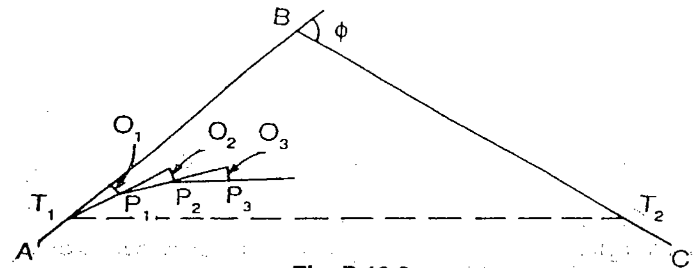


Fig. P.10.3

Solution

(a) Deflection angle $\phi = 180°0' - 126°48' = 53°12'$

(b) Tangent length = $R \tan \phi/2 = 300 \times \tan 26°36' = 150.23$ m

(c) Curve length = $\frac{\pi R\phi}{180°} = \frac{\pi \times 300 \times 53°12'}{180°} = 278.55$ m

(d) Chainage of T_1 = 510.23 – 150.23 = 360.00 m

(e) Chainage of T_2 = 360.00 + 278.55 = 638.55 m

Here no initial sub-chord is required as the chainage of T_1 is already a round number.

(f) Number of full chords (30 m) = 9

Chainage covered = 360.00 + 270.00 = 630.00 m

Length of final sub-chord = 638.55 – 630.00 = 8.55 m

First offset, $O_1 = \frac{b_1^2}{2R} = \frac{30^2}{2 \times 300} = 1.5$ m (here, b_1 to $b_9 = 30$ m, $b_{10} = 8.55$ m)

Second offset, $O_2 = \frac{b_2(b_1 + b_2)}{2R}$

$= \frac{2b_2^2}{2R} = \frac{b_2^2}{R}$ (as $b_1 = b_2$ = full chord)

$= \frac{30^2}{300} = 3$ m

Here, the second (O_2) through the ninth offsets (O_9) are of 3 m

Tenth offset,

$$O_{10} = \frac{8.55(30 + 8.55)}{2 \times 300} = 0.55 \text{ m}$$

Problem 4 The alignment of a road is as follows:

Line	WCB	Length (m)
AB	30°0′	250
BC	90°0′	150
CD	140°0′	325

These three lines are to be connected by a single circular curve. Find the radius and tangent length.

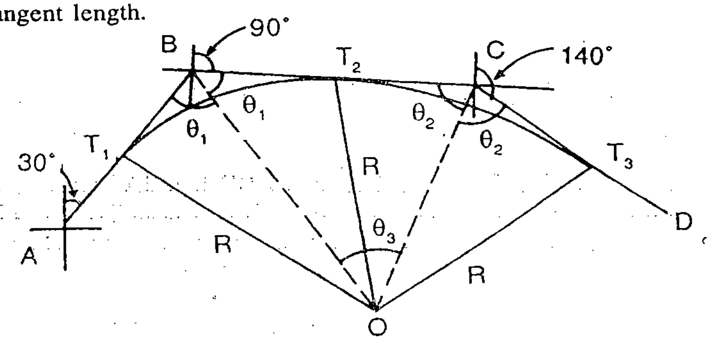


Fig. P.10.4

Solution

$\angle ABC$ = BB of AB − FB of BC = 210° − 90° = 120°

$$\theta_1 = \frac{120°}{2} = 60°$$

$\angle BCD$ = BB of BC − FB of CD

$= 270° - 140° = 130°$

$$\theta_2 = \frac{130°}{2} = 65°$$

$$\theta_3 = 180° - (60° + 65°) = 55°$$

Applying the sine rule in ΔOBC,

$$\frac{OB}{\sin\theta_2} = \frac{OC}{\sin\theta_1} = \frac{BC}{\sin\theta_3}$$

$$OB = BC \times \frac{\sin 65°}{\sin 55°}$$

$$= 150 \times \frac{\sin 65°}{\sin 55°} = 165.97 \text{ m}$$

$$OC = BC \times \frac{\sin 60°}{\sin 55°}$$

$$= 150 \times \frac{\sin 60°}{\sin 55°} = 158.59 \text{ m}$$

Radius R = OB sin 60°

= 165.97 × 0.866 = 143.73 m

or Radius R = OC × sin 65°

= 158.59 × 0.9063

= 143.73 m (checked)

Tangent length $BT_1 = BT_2$ = OB cos 60°

= 165.97 × 0.5 = 82.98 m

Tangent length $CT_2 = CT_3$ = OC × cos 65°

= 158.59 × 0.4226 = 67.02 m

Problem 5 Two straight lines AB and CD when produced intersect at a point I at an angle of 30°. The line BC makes an angle of 120° with CD. The length of BC is 320 m. It is proposed to introduce a reverse curve consisting of two circular arcs. The radius of one arc is 400 m and the chainage of B is 900 m. Calculate:

(a) The radius of the other arc,
(b) The lengths of the arcs, and
(c) The chainage of T_3.

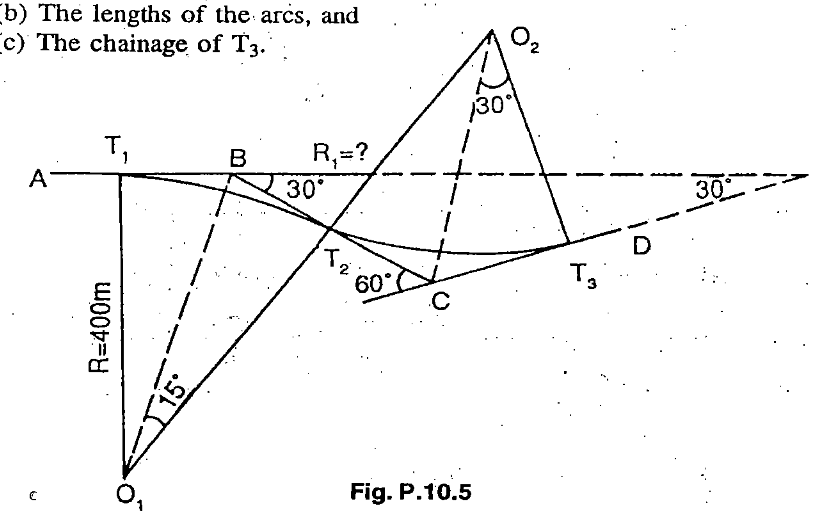


Fig. P.10.5

Solution

Given radius $O_1T_1(R)$ = 400 m

Let Radius $O_2T_2 = R_1$

BT_2 = 400 tan 15° = 107.18 m

∴ CT_2 = 320 − 107.18 = 212.82 m

Again $CT_2 = R_1$ tan 30°

$$R_1 = \frac{212.82}{\tan 0 \times 30°} = 368.62 \text{ m}$$

Length of arc,

$$T_1T_2 = \frac{\pi \times 400 \times 30°}{180°} = 209.43 \text{ m}$$

Length of arc,

$$T_2T_3 = \frac{\pi \times 368.62 \times 60°}{180°} = 368.02 \text{ m}$$

Chainage of T_1 = chainage of B − BT_1 ($BT_2 = BT_1$)
= 900.00 − 107.18 = 792.82

Chainage of T_3 = chainage of T_1 + arc T_1T_2 + arc T_2T_3
= 792.82 + 209.43 + 386.02
= 1,388.27 m

Problem 6 Two parallel lines 200 m apart are to be joined by a reverse curve with a deflection angle of 30°. If the radius of the first arc is 400 m and the chainage of the starting point of the curve 1,500 m, calculate the radius of the second arc, the chainage of the point of reverse curvature, and the finishing point of the reverse curve.

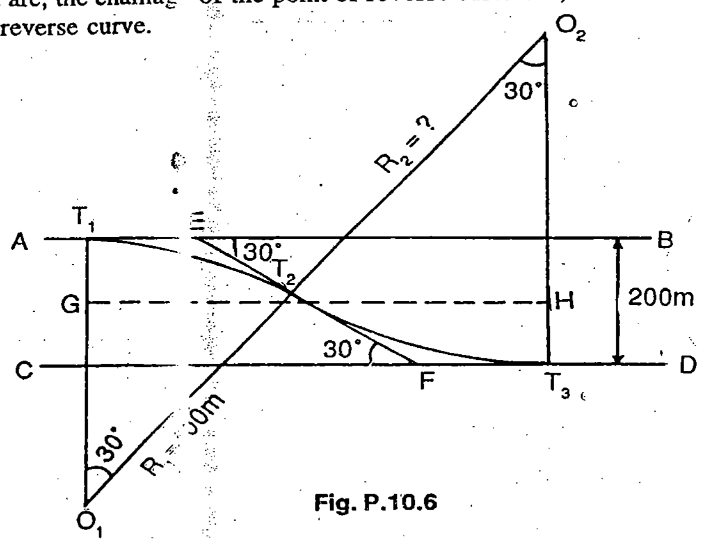


Fig. P.10.6

Solution
Given Data:
AB and CD are parallel lines 200 m apart.
EF is the common tangent.

T_1 = point of commencement (chainage = 1,500 m)
T_2 = point of reverse curvature
T_3 = finishing point

$$T_1G = O_1T_1 - O_1G = R_1 - R_1 \cos 30° = R_1(1 - \cos 30°)$$
$$= 400\ (1 - \cos 30°) = 53.6 \text{ m}$$

Similarly,

$$T_3H = R_2\ (1 - \cos 30°) = 0.134R_2$$

Here,

$$T_1G + T_3H = 200 \text{ m}$$
$$200 = 53.6 + 0.134R_2$$
$$R_2 = 1{,}092.5 \text{ m}$$

Length of arc $$T_1T_2 = \frac{\pi \times 400 \times 30°}{180°} = 209.44 \text{ m}$$

Length of arc $$T_2T_3 = \frac{\pi \times 1{,}092.5 \times 30°}{180°} = 572.03 \text{ m}$$

Chainage of point of reverse curvature (T_2) = chainage of T_1 + arc T_1T_2 = 1,500 + 209.44
= 1709.44 m

Chainage of finishing point T_3 = chainage of T_2 + arc T_2T_3
= 1,709.44 + 572.03
= 2281.47 m

Problem 7 Two straight lines AB and BC intersect at a point B of chainage 1,000 m. To avoid an obstacle, another line EF is taken to connect AB and BC, so that ∠AEF = 135° and ∠EFC = 145°. The radius of the first arc is 400 m and that of the second, 200 m. Calculate the chainages of:

(a) the tangent points, and
(b) the point of compound curvature,

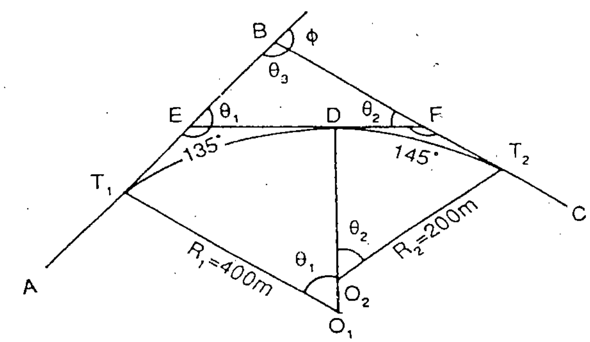


Fig. P.10.7

Solution T_1 and T_2 are the tangent points, and D is the point of compound curvature.

$$\theta_1 = 180° - 135° = 45°$$
$$\theta_2 = 180° - 145° = 35°$$
$$\theta_3 = 180° - (45 + 35°) = 100°$$

$$ET_1 = ED = R_1 \tan \frac{45°}{2} = 400 \times \tan 22.5° = 165.68 \text{ m}$$

$$FT_2 = FD = R_2 \tan \frac{35°}{2} = 200 \times \tan 17.5° = 63.05 \text{ m}$$

$$EF = ED + FD = 165.68 + 63.05 = 228.73 \text{ m}$$

Applying the sine rule in ΔBEF,

$$\frac{BE}{\sin \theta_2} = \frac{BF}{\sin \theta_1} = \frac{EF}{\sin \theta_3}$$

or $$BE = EF \times \frac{\sin \theta_2}{\sin \theta_3} = 228.73 \times \frac{\sin 35°}{\sin 100°} = 133.22 \text{ m}$$

$$BF = EF \times \frac{\sin \theta_1}{\sin \theta_3} = 228.73 \times \frac{\sin 45°}{\sin 100°} = 164.23 \text{ m}$$

$$\text{Chainage of } T_1 = 1{,}000 - (BE + ET_1)$$
$$= 1{,}000 - (133.22 + 165.68) = 701.10 \text{ m}$$

$$\text{Chainage of D} = 701.10 + \frac{\pi \times 400 \times 45°}{180°} = 1{,}015.25 \text{ m}$$

$$\text{Chainage of } T_2 = 1{,}015.25 + \frac{\pi \times 200 \times 35°}{180°} = 1{,}137.42 \text{ m}$$

Problem 8 Two straight lines AB and BC intersect at B, the chainage of B being 1,500.00 m. Another line EF intersects AB and BC such that $\angle BEF = 30°30'$ and $\angle BFE = 40°30'$. The length EF is 175 m. Find the radius of the curve which will be tangential to AB, EF and BC. Also calculate the chainages of the tangent points.

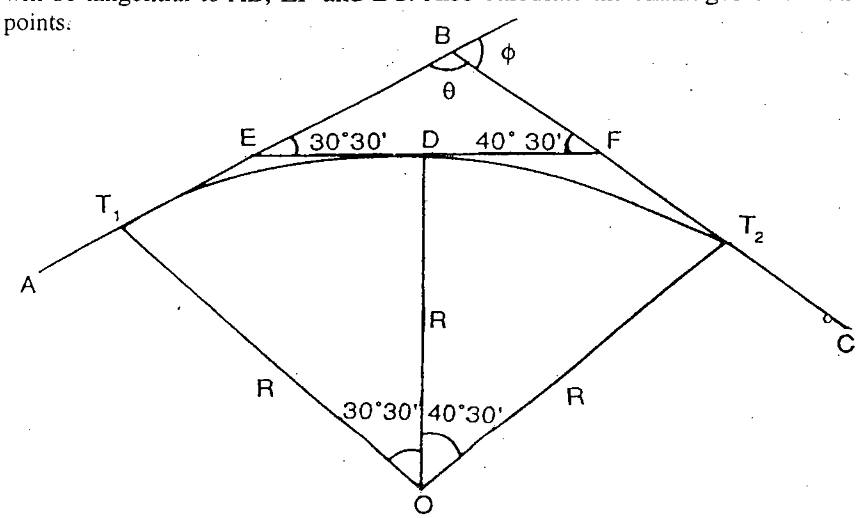


Fig. P.10.8

Solution

Let R be the radius of the curve.

$$ED = ET_1 = R \tan 15°15' = 0.2726\,R$$
$$FD = FT_2 = R \tan 20°15' = 0.3689\,R$$

Again $ED + FD = EF = 175$ m,

$$0.2726R + 0.3689R = 175 \text{ m}$$
$$0.6415R = 175$$
$$R = 272.79 \text{ m}$$

$$ET_1 = 0.2726 \times 272.79 = 74.36 \text{ m}$$
$$FT_2 = 0.3689 \times 272.79 = 100.63 \text{ m}$$

$$\text{Angle } \theta = 180° - (30°30' + 40°30') = (180° - 71°)$$

Applying the sine rule in ΔBEF,

$$\frac{BE}{\sin 40°30'} = \frac{BF}{\sin 30°30'} = \frac{EF}{\sin (180° - 71°)}$$

$$BE = EF \times \frac{\sin 40°30'}{\sin 71°} = 175 \times \frac{0.6494}{0.9455} = 120.19 \text{ m}$$

$$BF = 175 \times \frac{\sin 30°30'}{\sin 71°} = 93.93 \text{ m}$$

$$\text{Curve length } T_1D = \frac{\pi \times 272.79 \times 30°30'}{180°} = 145.21 \text{ m}$$

$$\text{Curve length } DT_2 = \frac{\pi \times 272.79 \times 40°30'}{180°} = 192.82 \text{ m}$$

$$\text{Chainage of } T_1 = 1{,}500 - (BE + ET_1)$$
$$= 1{,}500 - (120.19 + 74.36)$$
$$= 1{,}305.45 \text{ m}$$

$$\text{Chainage of } T_2 = 1{,}305.45 + 145.21 + 192.82 = 1{,}643.48 \text{ m}$$

Problem 9 Two tangents AB and BC meet at B, the deflection angle being 40°. It is proposed to connect AB and BC with a circular curve passing through a point P. The distance between P and B is 50 m, and the line BP makes an angle of 80° with AB. Find the radius of the curve.

Solution

Let R be the radius of the curve.

Here $OT_1 = R$

From ΔBOT_1,

$$\frac{OT_1}{OB} = \cos 20°$$

$$\therefore \quad OB = \frac{R}{\cos 20°} = 1.064\,R$$

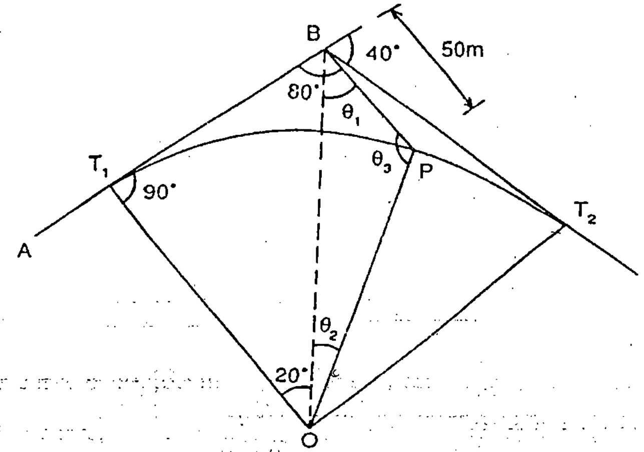


Fig. P.10.9

$$\theta_1 = 80° - 70° = 10° \qquad \left(\angle T_1BO = \frac{180° - 40°}{2} = 70°\right)$$

From Δ BOP,

$$\frac{OB}{\sin\theta_3} = \frac{OP}{\sin\theta_1} = \frac{BP}{\sin\theta_2}$$

or
$$\frac{1.064\,R}{\sin\theta_3} = \frac{R}{\sin\theta_1} = \frac{50}{\sin\theta_2}$$

or
$$\frac{1.064\,R}{\sin\theta_3} = \frac{R}{\sin 10°}$$

$\therefore$ $\sin\theta_3 = 1.064 \times \sin 10° = 0.1847$

$\therefore$ $\theta_3 = \sin^{-1} 0.1847 = 10.6°$

Again the value of $\theta_3 = 180° - 10.6° = 169.4°$

$\therefore$ $\theta_2 = 180° - (169.4° + 10°)$ $= 0.6°$ [As $\sin\theta° = \sin(180° - \theta°)$

$\sin 10.6° = \sin(180° - 10.6°)$

$\therefore$ $\sin\theta_3 = \sin 169.4°$

$\therefore$ $\theta_3 = 169.4°$]

$\therefore$
$$\frac{R}{\sin 10°} = \frac{50}{\sin 0.6°}$$

$\therefore$
$$R = \frac{50 \times \sin 10°}{\sin 0.6°} = 829.12 \text{ m}$$

Problem 10 A composite curve is to be set out with the following data: Deflection angle = 60°, maximum speed of vehicle = 80 km/hr, centrifugal ratio = 1/8, rate of change of radial acceleration = 0.3 m/s^3, chainage of intersection point = 1,150 m. Calculate: (a) Radius of circular curve,

(b) Length of transition curve,
(c) Chainages of tangent points and junctions of transition curve with circular curve.

Solution

$$\text{Speed} = \frac{80 \times 1{,}000}{60 \times 60} = 22.22 \text{ m/s}$$

$$\text{Centrifugal ratio} = \frac{V^2}{gR} = \frac{1}{8} \quad \text{or} \quad R = \frac{V^2 \times 8}{g}$$

$$R = \frac{(22.22)^2 \times 8}{9.81} = 402.63 \text{ m}$$

(a) Radius of circular curve = 402.63 m

(b) Length of transition curve, $L = \dfrac{V^3}{K \times R}$ $(K = 0.3 \text{ m/s}^3)$

$$= \frac{(22.22)^3}{0.3 \times 402.63} = 90.82 \text{ m}$$

$$\text{Spiral angle } \phi = \frac{L}{2R} \text{ radians} = \frac{90.82 \times 180}{2 \times 402.63 \times \pi} = 6.46°$$

$$\text{Central angle} = \Delta - 2\phi = 60° - 6.46° \times 2 = 47.08°$$

$$\text{Length of circular curve} = \frac{\pi \times 402.63 \times 47.08°}{180°} = 330.84 \text{ m}$$

$$\text{Shift of the curve } (s) = \frac{L^2}{24R} = \frac{(90.82)^2}{24 \times 402.63} = 0.85 \text{ m}$$

$$\text{Tangent length } T = (R + s)\tan\frac{\Delta}{2} + \frac{L}{2}$$

$$= (402.63 + 0.85)\tan 30° + 45.41 = 278.35 \text{ m}$$

(c) Chainage of 1st tangent point = 1,150 − 278.35 = 871.65 m
Chainage of 1st junction = 871.65 + 90.82 = 962.47 m
Chainage of 2nd junction = 962.47 + 330.84 = 1,293.31 m
Chainage of 2nd tangent point = 1,293.31 + 90.82 = 1,384.13 m

Problem 11 Tabulate the data required for setting out a curve by the deflection angle method, considering the following information:

(a) Angle of intersection = 145°
(b) Chainage of point of intersection = 1,580 m
(c) Degree of curve = 5°
(d) Least count of theodolite = 20″
(e) Peg interval = 30 m

Solution Deflection angle = 180° − 145° = 35°

$$\text{Radius} = \frac{1{,}719}{5} = 343.8 \text{ m}$$

(a) Tangent length $= R \tan \frac{35^\circ}{2} = 343.8 \times \tan 17.5^\circ = 108.39$ m

(b) Curve length $= \frac{\pi R \phi}{180^\circ} = \frac{\pi \times 343.8 \times 35^\circ}{180^\circ} = 210.0$ m

(c) Chainage of 1st tangent point = 1,580 – 108.39 = 1,471.61 m

(d) Chainage of 2nd tangent point = 1,471.61 + 210.0 = 1,681.61 m

(e) Length of initial sub-chord = 1,480 – 1,471.61 = 8.39 m

(f) Number of full chords of length 30 m = 6
Chainage covered = 1,480 + 180 = 1,660 m

(g) Length of final sub-chord = 1,681.61 – 1,660.00 = 21.61 m

(h) Deflection angle for initial sub-chord $= \frac{1{,}781.9 \times 8.39}{343.8} = 0^\circ41'57''$

(i) Deflection angle for full chord $= \frac{1{,}781.9 \times 30}{343.8} = 2^\circ29'59''$

(j) Deflection angle for final sub-chord $= \frac{1{,}718.9 \times 21.61}{343.8} = 1^\circ48'3''$

(k) Arithmetical check:

Total deflection angle $= \phi/2 = \frac{35^\circ}{2} = 17^\circ30'$

Here,
Total deflection angle $= 0^\circ41'57'' + 6 \times 2^\circ29'59'' + 1^\circ48'3'' = 17^\circ29'54''$
$= 17^\circ30'$ (say)

So, the calculation of deflection angles is correct.

(l) Data for field check:

(i) Apex distance $= R(\sec \phi/2 - 1)$
$= 343.8 (\sec 17^\circ30' - 1) = 16.69$ m

(ii) Versed sine of curve $= R(1 - \cos \phi/2)$
$= 343.8 (1 - \cos 17^\circ30') = 15.92$ m

(m) Setting out table

Point	Chainage	Chord length	Deflection for chord	Total deflection angle	Angle to be set	Remark
T_1	1,471.61	—	—	—	—	Starting point of curve
P_1	1,480	8.39	0°41′57″	0°41′57″	0°42′0″	
P_2	1,510	30	2°29′59″	3°11′56″	3°12′0″	
P_3	1,540	30	2°29′59″	5°41′55″	54°42′0″	
P_4	1,570	30	2°29′59″	8°11′54″	8°12′0″	
P_5	1,600	30	2°29′59″	10°41′53″	10°42′0″	
P_6	1,630	30	2°29′59″	13°11′52″	13°12′0″	
P_7	1,660	30	2°29′59″	15°41′51″	15°42′0″	
T_2	1,681.61	21.61	1°48′3″	17°29′54″	17°30′0″	Finishing point of curve

10.14 WORKED OUT PROBLEMS ON VERTICAL CURVES

Problem 1 Along the alignment of a road, it is found in a particular portion that a grade of + 0.5% is followed by one of – 0.7%. The two ends of this portion are to be connected by a parabolic vertical curve. The chainage and RL of the intersection point are 550.00 and 375.50 m respectively. Calculate the RLs of the points on the curve, taking a peg interval of 20 m. The rate of change of grade is 0.1% per 20 m. Tabulate the result.

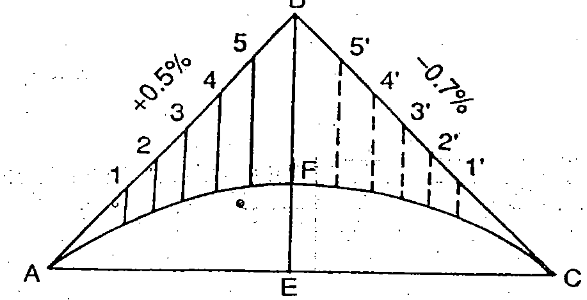


Fig. P.10.10

Solution

Given Data:

Chainage of B = 550.00 m (apex)
RL of B = 375.50 m
$g_1 = +0.5\%$ $g_2 = -0.7\%$

Rate of change of grade, $r = 0.1\%$ per 20 m

(a) Length of vertical curve $= \frac{0.5 - (-0.7)}{0.1} \times 20 = \frac{1.2}{0.1} \times 20 = 240$ m

The length of the curve on either side of the apex is taken as 120 m.

(b) Chainage of A = 550.00 – 120.00 = 430.00 m

(c) Chainage of C = 430 + 240 = 670.00 m

(d) RL of A $= 375.50 - \frac{0.5 \times 120}{100} = 374.90$ m

(e) RL of C $= 375.50 - \frac{0.7 \times 120}{100} = 374.66$ m

(f) RL of E = 1/2 (374.90 + 374.66) = 374.78 m

(g) RL of F = 1/2 (375.50 + 374.78) = 375.14 m (vertex)

(h) Tangent correction at the apex = 375.50 – 375.14 = 0.36 m

Tangent corrections are found out at 20 m intervals, from the relation:

$$y = \frac{g_1 - g_2}{400 \times l} \times x^2 \qquad (l = \text{half-curve length})$$

(i) Tangent correction for 1st point $= \frac{1.2}{400 \times 120} \times (20)^2 = 0.01$ m

$$\text{Tangent correction for 2nd point} = \frac{1.2}{400 \times 120} \times (40)^2 = 0.04 \text{ m}$$

$$\text{Tangent correction for 3rd point} = \frac{1.2}{400 \times 120} \times (60)^2 = 0.09 \text{ m}$$

$$\text{Tangent correction for 4th point} = \frac{1.2}{400 \times 120} \times (80)^2 = 0.16 \text{ m}$$

$$\text{Tangent correction for 5th point} = \frac{1.2}{400 \times 120} \times (100)^2 = 0.25 \text{ m}$$

$$\text{Tangent correction for 6th point (apex)} = \frac{1.2}{400 \times 120} \times (120)^2 = 0.36 \text{ m} \quad \text{(checked)}$$

Here tangent correction is to be subtracted.

(j) RL of the points on the grade (considering left-hand side of curve),

$$\text{Rise per 20 m} = \frac{0.5 \times 20}{100} = 0.10 \text{ m}$$

Here,

RL of 1st point on grade = (RL of A + 0.10)

∴ RL of 1st point on grade = 374.90 + 0.10 = 375.00 m
RL of 2nd point on grade = 375.00 + 0.10 = 375.10 m
RL of 3rd point on grade = 375.10 + 0.10 = 375.20 m
RL of 4th point on grade = 375.20 + 0.10 = 375.30 m
RL of 5th point on grade = 375.30 + 0.10 = 375.40 m
RL of 6th point on grade = 375.40 + 0.10 = 375.50 m (apex point)

(k) RLs of the points on the curve (considering the left-hand side of the curve):

RL of 1st point on curve = 375.00 – 0.01 = 374.99 m
RL of 2nd point on curve = 375.10 – 0.04 = 375.06 m
RL of 3rd point on curve = 375.20 – 0.09 = 375.11 m
RL of 4th point on curve = 375.30 – 0.16 = 375.14 m
RL of 5th point on curve = 375.40 – 0.25 = 375.15 m
RL of 6th point on curve vertex (F) = 375.50 – 0.36 = 375.14 m (checked)

The curve is assumed to be symmetrical, and so the tangent correction for the right side are exactly the same as for the left side. Only the RL of the point on the grade and those of the points on the curve will be changed as the gradient is different.

(l) RLs of the points on the grade on the right side:

$$\text{Fall per 20 m} = \frac{0.7 \times 20}{100} = 0.14 \text{ m}$$

RL of 5th point on grade = RL of B – 0.14 = 375.50 – 0.14 = 375.36 m
RL of 4th point on grade = 375.36 – 0.14 = 375.22 m

RL of 3rd point on grade = 375.22 – 0.14 = 375.08 m
RL of 2nd point on grade = 375.08 – 0.14 = 374.94 m
RL of 1st point on grade = 374.94 – 0.14 = 374.80 m
RL of point C = 374.80 – 0.14 = 374.66 m (checked)

(m) RL of the points on the curve on the right-hand side are shown in the setting out table.

Setting out table

Point	Chainage	Grade RL	Tangent correction (–ve)	Curve RL	Remark
A	430.00	374.90	0.00	374.90	Starting point on curve
1	450.00	375.00	0.01	374.99	
2	470.00	375.10	0.04	375.06	
3	490.00	375.20	0.09	375.11	
4	510.00	375.30	0.16	375.14	
5	530.00	375.40	0.25	375.15	
6 (B)	550.00	375.50	0.36	375.14	Vertex of curve
5′	570.00	375.36	0.25	375.11	
4′	590.00	375.22	0.16	375.06	
3′	610.00	375.08	0.09	374.99	
2′	630.00	374.94	0.04	374.90	
1′	650.00	374.80	0.01	374.79	
C	670.00	374.66	0.00	374.66	End of curve

Problem 2 In a road alignment a grade of – 1.0% is followed one of + 0.5%. The chainage and RL of the intersection point are 400 and 250.50 m respectively. The rate of change of grade is 0.1% per 20 m. Calculate the necessary data for setting out the vertical curve, taking a peg interval of 30 m.

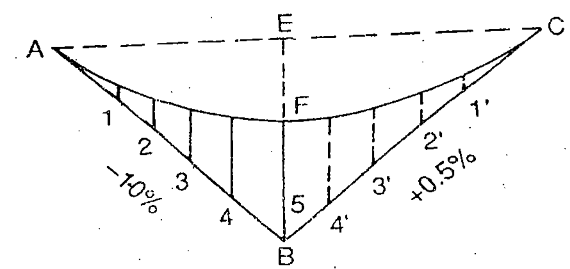


Fig. P.10.11

Solution

Given data:

Chainage of B = 400.00 m
RL of B = 250.50 m

$g_1 = -1.0\%$, $g_2 = +0.5\%$
$\gamma = 0.1\%$ per 20 m

(a) Length of vertical curve $= \dfrac{-1.0-(+0.5)}{0.1} \times 20 = \dfrac{-1.5}{0.1} \times 20$

$= 300$ m (–ve sign has no significance)

The length of curve on either side of the apex is taken as 150 m.

(b) Chainage of A = chainage of B – 150.00
= 400.00 – 150.00 = 250.00 m

(c) Chainage of C = 250.00 + 300.00 = 550.00 m

(d) RL of A = RL of B + $\dfrac{1 \times 150}{100}$ = 250.50 + 1.50 = 252.00 m

(e) RL of C = RL of B + $\dfrac{0.5 \times 150}{100}$
= 250.50 + 0.75 = 251.25 m

(f) RL of E = 1/2 (252.00 + 251.25) = 251.625 m

(g) RL of F = 1/2 (250.50 + 251.625) = 251.06 m
A peg interval of 30 m is taken.

(h) RL on the grade on side AB (from A towards B):

$$\text{Fall per 30 m} = \frac{1 \times 30}{100} = 0.30 \text{ m}$$

RL of 1st point = 252.00 – 0.30 = 251.70 m (RL of A – 0.30)
RL of 2nd point = 251.70 – 0.30 = 251.40 m
RL of 3rd point = 251.40 – 0.30 = 251.10 m
RL of 4th point = 251.10 – 0.30 = 250.80 m
RL of point B = 250.80 – 0.30 = 250.50 m (checked)

(i) RL on the grade on side BC (from B towards C):

$$\text{Rise per 30 m} = \frac{0.5 \times 30}{100} = 0.15 \text{ m}$$

RL of 4th point = RL of B + 0.15
= 250.50 + 0.15 = 250.65 m
RL of 3rd point = 250.65 + 0.15 = 250.80 m
RL of 2nd point = 250.80 + 0.15 = 250.95 m
RL of 1st point = 250.95 + 0.15 = 251.10 m
RL of C point = 251.10 + 0.15 = 251.25 m (checked)

(j) Tangent correction from expression,

$$y = \frac{g_1 - g_2}{400 \times l} \times x^2 \quad (l = \text{half length of curve})$$

Tangent correction for 1st point $y_1 = \dfrac{-1.0-0.5}{400 \times 150} \times (30)^2$

$$= \frac{-1.5 \times (30)^2}{400 \times 150} = -0.02 \text{ m}$$

Tangent correction for 2nd point $y_2 = \dfrac{-1.5 \times (60)^2}{400 \times 150} = -0.09$ m

Tangent correction for 3rd point $y_3 = \dfrac{-1.5 \times (90)^2}{400 \times 150} = -0.20$ m

Tangent correction for 4th point $y_4 = \dfrac{-1.5 \times (120)^2}{400 \times 150} = -0.36$ m

Tangent correction for (BF) $= \dfrac{-1.5 \times (150)^2}{400 \times 150} = -0.56$ m

Check: RL of F – RL of B = 251.06 – 250.50 = 0.56 m
Here, the negative sign has no significance.

The tangent correction is to be added to the RL of the grade to get the RL on the curve.

Table for setting out

Point	Chainage	Grade RL	Tangent correction (+ve)	Curve RL	Remark
A	250.00	252.00	0.00	252.00	Starting point of the curve
1	280.00	251.70	0.02	251.72	
2	310.00	251.40	0.09	251.49	
3	340.00	251.10	0.20	251.30	
4	370.00	250.80	0.36	251.16	
Vertex (B)	400.00	250.50	0.56	251.06	Vertex of the curve
4′	430.00	250.65	0.36	251.01	
3′	460.00	250.80	0.20	251.00	
2′	490.00	250.95	0.09	251.04	
1′	520.00	251.10	0.02	251.12	
C	550.00	251.25	0.00	251.25	End of curve

SHORT QUESTIONS WITH ANSWERS FOR VIVA

Q. 1 What are the different types of horizontal curves?
Ans. The types are: simple circular, compound, reverse, transition and combined.

Q. 2 What is the degree of a curve?
Ans. The angle subtended at the centre by the unit chord (of 30 m) of a curve is known as the degree of that curve.

Q. 3 What is a 5° curve?
Ans. When the unit chord (30 m) subtends an angle of 5° at the centre, the corresponding curve is known as a 5° curve.

Q. 4 What is the radius of a 1° curve?
Ans. The radius of a 1° curve is 1,719 m.

Q. 5 What is the relation between the radius and degree of a curve?
Ans. Radius $= \dfrac{1{,}719}{D}$ where D = degree of curve

Q. 6 What do the terms, 'rear tangent' and 'forward tangent' mean?

Ans. Along the direction of progress of a survey, the first tangent is known as the rear or back tangent and the second as the forward tangent.

Q. 7 Define the terms 'point of curve' and 'point of tangency'

Ans. Along the direction of progress of a survey, the first tangent point is called the point of curve and the second such point as the point of tangency.

Q. 8 What is apex distance? Express it mathematically.

Ans. The distance between the vertex and apex or summit of a curve is known as the apex distance of the curve.

$$\text{Apex distance} = R (\sec \phi/2 - 1)$$
$$R = \text{radius of curve}$$
$$\phi = \text{deflection angle}$$

Q. 9 What is the versed sine of a curve? Express it mathematically.

Ans. The distance between the apex of a curve and the mid-point of a long chord is known as the versed sine of the curve.

$$\text{Versed sine} = R (1 - \cos \phi/2).$$

Q. 10 What are the different methods of curve setting?

Ans. There are mainly two methods:

(a) The chain-and-tape method, and
(b) The instrumental method.

Again, the chain and tape method may involve:

(a) offsets from the long chord,
(b) offsets from the chord produced,
(c) offsets from tangents, and
(d) successive bisection of arcs.

Instrumental methods may involve the use of:

(a) a single theodolite, and
(b) Two theodolites.

Q. 11 What is superelevation? Why it is provided?

Ans. In a circular path, the outer edge is raised above the inner edge. This process of raising the outer edge is known as superelevation.

When a vehicle passes from a straight to a curved path, then a centrifugal is developed which tends to push it out of the track. When superelevation is provided, the vehicle get inclined and the sine component of the weight of the vehicle counterbalances the centrifugal force.

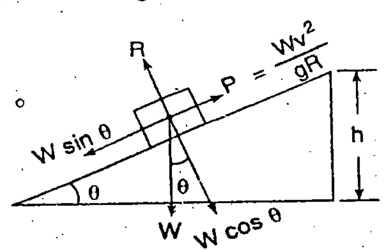


Fig. P. 10.12

According to this condition,

$$W \sin \theta = \frac{Wv^2}{gR}$$

h = superelevation, which is given by $\frac{bv^2}{gR}$ for roads, and $\frac{Gv^2}{gR}$ for railways.

Q. 12 What is centrifugal ratio?

Ans. The ratio of the centrifugal force and the weight of the vehicle is known as centrifugal ratio.

$$CR = \frac{V^2}{gR}$$

Q. 13 What is a transition curve? Where is such a curve provided?

Ans. A curve of varying radius is known as a transition curve.
It is provided in railway tracks at either end of a circular curve.

Q. 14 What is the shift of a curve?

Ans. When a transition curve is introduced in a circular curve, then the circular curve is found to be shifted by some distance towards the centre. This distance is known as the shift of the curve.

Q. 15 What is a spiral angle?

Ans. The angle between the initial tangent and the common tangent at the point of junction of the transition and circular curves is known as the spiral angle.

Q. 16 What is the difference between a compound and a composite curve?

Ans. A compound curve is a combination of two circular curves of different radii.
A composite curve is a combination of a transition and a circular curve.

Q. 17 Two parallel lines are to be connected. What type of curve would you suggest?

Ans. A reverse curve should be adopted.

Q. 18 State any expression for finding the length of a transition curve.

Ans. $$\text{Length of transition curve} = \frac{V^3}{CR}$$

where V = speed in metres per second
R = radius of curve in metres
C = rate of change of radial acceleration in metres per second cube

Q. 19 What are initial and final sub-chords?

Ans. When the chainage of the first tangent point of a curve is an odd figure, then some length is provided to make this chainage a round figure. The length provided is known as the initial sub-chord.

After the initial sub-chord, a number of full chords (20 m or 30 m) are provided. At the end of the curve, a fractional length may be required to reach the second tangent point. This length is known as the final sub-chord.

Q. 20 What are the different types of vertical curves?

Ans. Vertical curves are of two types—summit curves and sag curves. They are called summit curve when

(a) An upgrade is followed by a downgrade.
(b) A downgrade is followed by another downgrade.

They are called sag curves when:

(i) A downgrade is followed by an upgrade.
(ii) An upgrade is followed by another upgrade.

Q. 21 How is gradient expressed?
Ans. Gradient may be expressed: (a) as a percentage, e.g. 1%, 2%, etc. (b) As 1 in n, e.g. 1 in 100, 1 in 200, etc.
Q. 22 On what basis is a vertical curve designed?
Ans. A vertical curve is designed on the basis of sight distance.
Q. 23. What is the type of vertical curve preferred?
Ans. A parabolic curve is preferred, because it is easy for a vehicle to pass over. Also, it is simple to provide minimum sight distance in the case of such a curve.
Q. 24 What is tangent correction?
Ans. While constructing a vertical curve, cutting or banking has to be done to obtain the desired shape. The height of cutting or banking is known as tangent correction.
Q. 25 What are the possible different shapes of transition curves?
Ans. A transition curve may be of the shape of:
(a) Euler's spiral
(b) A cubical spiral
(c) A cubic parabola
(d) Bernoulli's lemniscate curve

EXERCISES

1. (a) Why is a curve provided?
(b) What is the degree of a curve?
(c) Derive a relation between the radius and degree of a curve.
2. What are the different types of curves?
Draw neat sketches of each.
3. Draw a neat sketch of a circular curve and show the following notation thereon: (a) Back tangent, (b) forward tangent, (c) point of commencement, (d) point of tangency, (e) point of intersection, (f) angle of deflection, (g) angle of intersection, (h) long chord, (i) apex distance, and (j) versed sine of curve.
4. Describe how you would set a circular curve by the method of offsets from the long chord with the help of chain and tape.
5. Describe the method of setting a circular curve by perpendicular offsets from the tangent with the help of chain and tape.
6. Describe the method of setting a simple circular curve by Rankine's deflection angle method.
7. (a) Explain why superelevation is required in roads and railways.
(b) What is a transition curve?
(c) Why and where are transition curves provided?
8. State the different methods of calculating the length of a transition curve.
9. What is shift? Prove that a transition curve bisects a shift and that a shift bisects a transition curve.
10. Derive an expression for an ideal transition curve.
11. Explain the different methods of overcoming the difficulties in setting out circular curves.
12. (a) What is a vertical curve?
(b) Why is it provided?
(c) State an expression for calculating the length of a vertical curve.
(d) Show, with neat sketches, the different types of vertical curves possible.
13. Two straight lines T_1P and PT_2 are intersected by a third line AB, such that ∠PAB = 46°24′, ∠PBA = 32°36′ and the distance AB = 312 m. Calculate the radius of the simple circular curve which will be tangential to the three lines T_1P, AB and PT_2 and the chainages of the point of curve (T_1) and point of tangency (T_2), if the chainage of the point P is 2,857.5 m. (AMIE, Summer 1986)
(Ans. R = 432.68 m, chainage of T_1 = 2,500.8 m, chainage of T_2 = 2,933.48 m)
14. A road bend which deflects by 80° is to be designed for a maximum speed of 120 km/hr, a maximum centrifugal ratio of 1/4, and a maximum rate of change of radial acceleration of 30 cm/s^3. The curve should consist of a circular arc combined with two cubic spirals. Calculate:
(a) The radius of the circular arc,
(b) The requisite length of the transition curve, and
(c) The total length of the composite curve.
(Ans. R = 453.05 m l = 272.42 m L = 905.51 m)
15. Choose the correct alternative for questions (i) through (vi).
(i) The radius of a one-degree curve is
(a) 1,719 m (b) 1,760 m (c) 2,000 m
(ii) The relation between the radius (R) of and degree (D) of a curve is
(a) $R = \frac{D}{1{,}719}$ (b) $R = \frac{1{,}719}{D}$ (c) $\frac{R}{D} = 1{,}719$
(iii) With notations carrying their usual meanings, the superelevation (h) for railways is given by the relation
(a) $h = \frac{gv^2}{GR}$ (b) $h = \frac{gR^2}{GV}$ (c) $h = \frac{GV^2}{gR}$
(iv) The allowable centrifugal ratio (CR) for railways is
(a) $\frac{1}{4}$ (b) $\frac{1}{8}$ (c) $\frac{1}{6}$
(v) The first tangent point is also known as the
(a) Point of curve (b) Point of start (c) Point of tangent
(vi) The second tangent point is also known as the
(a) End point (b) Point of tangency (c) Rear point
(vii) When two tangents AB and BC meet at B, the point B is known as the
(a) Summit (b) Apex (c) Vertex
(xiii) Apex distance is given by
(a) $R\left(\sec\frac{\phi}{2} - 1\right)$ (b) $R\left(\cos\frac{\phi}{2} - 1\right)$ (c) $R\left(\sin\frac{\phi}{2} - 1\right)$
(ix) The versed sine of a curve is given by
(a) $R\left(1 - \sin\frac{\phi}{2}\right)$ (b) $R\left(1 - \cos\frac{\phi}{2}\right)$ (c) $R\left(1 - \cot\frac{\phi}{2}\right)$
(x) The length of a unit chord should not be more than
(a) $\frac{1}{20}$ th of the radius
(b) $\frac{1}{10}$ th of the radius
(c) $\frac{1}{40}$ th of the diameter
(xi) The length of a long chord is given by the expression
(a) $L = 2R\sin\frac{\phi}{2}$ (b) $L = 2R\tan\frac{\phi}{2}$ (c) $L = 2R\cos\frac{\phi}{2}$

(xii) The intrinsic equation of an ideal transition curve is given by the expression
(a) $\phi = \frac{L^2}{2R}$ degrees (b) $\phi = \frac{L}{2R^2}$ minutes (c) $\phi = \frac{L}{2R}$ radians

(xiii) An ideal transition curve is also known as a
(a) clothoid curve (b) cubical curve (c) parabolic curve

(xiv) A vertical curve is considered as a/an
(a) elliptical curve (b) parabolic curve (c) circular curve

(xv) The length of a transition curve is given by the relation
(a) $L = \frac{v^3}{cR}$ (b) $L = \frac{v^2}{cR}$ (c) $L = \frac{v}{cR}$

(xvi) A vertical curve is designed on the basis of the
(a) radius of the curve
(b) minimum sight distance
(c) change of gradient.

ANSWERS

15. (i) a (ii) b (iii) c (iv) b (v) a
(vi) b (vii) c (viii) a (ix) b (x) a
(xi) a (xii) c (xiii) a (xiv) b (xv) a
(xvi) b

11 Tacheometric Surveying

11.1 INTRODUCTION

Tacheometry is a branch of surveying in which horizontal and vertical distances are determined by taking angular observations with an instrument known as a tacheometer. The chaining operation is completely eliminated in such survey. Tacheometric surveying is adopted in rough and difficult terrain where direct levelling and chaining are either not possible or very tedious. It is also used in location survey for railways, roads, reservoirs, etc. Though not very accurate, tacheometric surveying is very rapid, and a reasonable contour map can be prepared for investigation works within a short time on the basis of such survey.

1. Instruments used in tacheometry

(a) The Tacheometer It is nothing but a transit theodolite fitted with a stadia diaphragm and an anallatic lens. Figure 11.1 shows the different forms of stadia diaphragm commonly used:

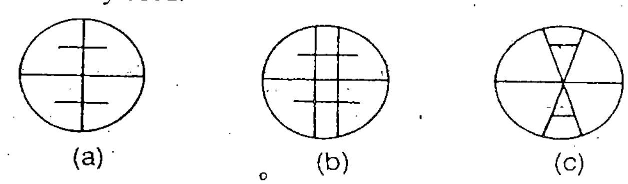


Fig. 11.1

(b) The Levelling Staff and Stadia Rod For short distances, ordinary levelling staves are used. The levelling staff is normally 4 m long, and can be folded into three parts. The graduations are so marked that a minimum reading of 0.005 or 0.001 m, can be taken.

For long sights a specially designed graduated rod is used, which is known as a stadia rod. It is also 4 m long, and may be folded or telescopic. The graduations are comparatively bold and clear and the minimum reading that can be taken is 0.001 m.

2. Characteristics of tacheometer

(a) The value of the multiplying constant f/i should be 100.
(b) The telescope should be powerful, having a magnification of 20 to 30 diameters.

(c) The aperture of the objective should be of a 35 to 45 mm diameter for there to be a bright image.

(d) The telescope should be fitted with an anallatic lens to make the additive constant $(f + d)$ exactly equal to zero.

(e) The eye-piece should be of greater magnifying power than usual, so that it is possible to obtain a clear staff reading from a long distance.

3. Principle of tacheometry The principle of tacheometry is based on the property of isosceles triangles, where the ratio of the distance of the base from the apex and the length of the base is always constant.

In Fig. 11.2, $o_1a_1a_2$, $o_1b_1b_2$, and $o_1c_1c_2$ are all isosceles triangles where D_1, D_2 and D_3 are the distances of the bases from the apices, and S_1, S_2 and S_3 are the lengths of the bases (staff intercepts).

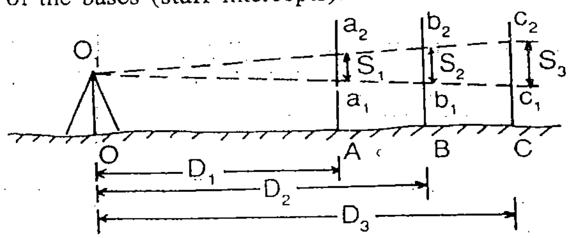


Fig. 11.2

So, according to the stated principle,

$$\frac{D_1}{S_1} = \frac{D_2}{S_2} = \frac{D_3}{S_3} = \frac{f}{i} \qquad \text{(constant)}$$

The constant f/i is known as the multiplying constant,

where, f = focal length of objective and i = stadia intercept

11.2 THEORY OF STADIA TACHEOMETRY

The following is the notation used in stadia tacheometry (Fig. 11.3):

O = optical centre of object glass

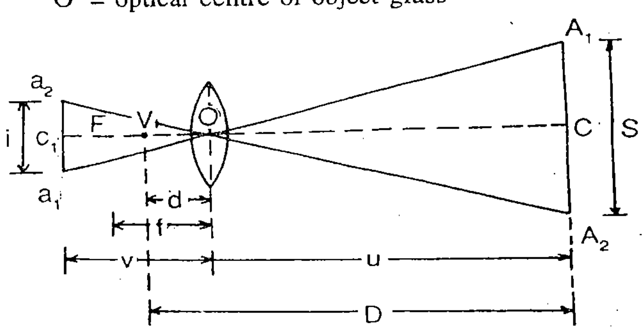


Fig. 11.3

A_1, A_2, C = readings on staff cut by three hairs

a_1, a_2, C = bottom, top and central hairs of diaphragm

$a_1a_2 = i$ = length of image

$A_1A_2 = S$ = staff intercept

F = focus

V = vertical axis of instrument

f = focal length of object glass

d = distance between optical centre and vertical axis of instrument

u = distance between optical centre and staff

v = distance between optical centre and image

From similar triangles a_1oa_2 and A_1OA_2, $\frac{i}{s} = \frac{v}{u}$

or $$v = \frac{iu}{s} \qquad (1)$$

From the properties of lenses,

$$\frac{1}{v} + \frac{1}{u} = \frac{1}{f} \qquad (2)$$

Putting the value of v in Eq. (2),

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

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

or $$\frac{1}{u}\left(\frac{s}{i} + 1\right) = \frac{1}{f}$$

or $$u = \left(\frac{s}{i} + 1\right)f \qquad (3)$$

But $$D = u + d$$

so, $$D = \left(\frac{s}{i} + 1\right) f + d$$

$$= \frac{s}{i} \times f + f + d = \left(\frac{f}{i}\right) \times s + (f + d)$$

The quantities (f/i) and $(f + d)$ are known as tacheometric constants. (f/i) is called the multiplying constant, as already stated, and $(f + d)$ the additive constant.

By adopting an anallatic lens in the telescope of a tacheometer, the multiplying constant is made 100, and the additive constant zero.

However, in some tacheometers the additive constants are not exactly zero, but vary from 30 cm to 60 cm (which are generally mentioned in the catalogue supplied by the manufacturer).

11.3 DETERMINATION OF TACHEOMETRIC OR STADIA CONSTANT

The constants may be determined by

1. Laboratory measurement
2. Field measurement

1. Laboratory measurement The focal length f of the lens can be determined by means of an optical bench, according to the equation:

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

The stadia intercept i can be measured from the diaphragm with the help of a vernier calliper.

The distance d between the optical centre and the vertical axis of the instrument can also be measured.

In this manner, the multiplying (f/i) and additive ($f + d$) constants can be calculated.

2. Field measurement

(a) A fairly level ground is selected. The tacheometer is set up at O and pegs are fixed at A_1, A_2 and A_3 known distances apart (see Fig. 11.4).

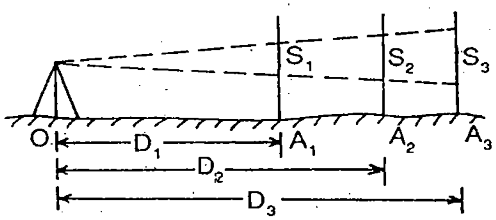


Fig. 11.4

(b) The staff intercepts (stadia hair readings) are noted at each of the pegs. Let these intercepts be S_1, S_2 and S_3.

(c) The horizontal distances of the pegs from O are accurately measured. Let these distances be D_1, D_2 and D_3.

(d) By substituting the values of D_1, D_2, ... and S_1, S_2, ... in the general equation

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

we get a number of equations, as follows:

$$D_1 = \left(\frac{f}{i}\right) S_1 + (f + d)$$

$$D_2 = \left(\frac{f}{i}\right) S_2 + (f + d) \quad \text{and so on.}$$

(e) By solving the equations in pairs, several values of (f/i) and ($f + d$) are obtained. The mean of these values gives the required constant.

Example Determine the values of stadia constants from the following observations.

Instrument station	Staff reading on	Distance (m)	Stadia readings	
			Lower	Upper
O	A	150	1.255	2.750
	B	200	1.000	3.000
	C	250	0.750	3.255

Solution We know from the theory of stadia tacheometry that

$$D = \left(\frac{f}{i}\right) \times s + (f + d) \quad (1)$$

Let $\frac{f}{i} = x$

and $(f + d) = y$

so, $D = x \cdot S + y$ (2)

In the first case, $150 = x(2.750 - 1.255) + y$

or $150 = 1.495x + y$ (3)

In the second case, $200 = x(3.000 - 1.000) + y$

or $200 = 2x + y$ (4)

In the third case, $250 = x(3.255 - 0.750) + y$

or $250 = 2.505x + y$ (5)

First set

Solving Eqs (3) and (4) $1.495x + y = 150$

$2x + y = 200$

Subtracting, we get $-0.505x = -50$

or $x = 99.00$

From Eq. (4) $y = 200 - 2 \times 99 = 2$

Second Set

Solving Eqs (4) and (5), $2x + y = 200$

$2.505x + y = 250$

Subtracting, we get $-0.505x = -50$

$\therefore$ $x = 99.00$

$y = 200 - 2 \times 99 = 2$

Third set

Solving Eqs (3) and (5), $1.495x + y = 150$

$2.505x + y = 250$

Subtracting, we get

$$-1.01x = -100$$

$$\therefore \quad x = 99.00$$

$$y = 2$$

So, $$\text{Average value of } x = \frac{1}{3}(99.00 + 99.00 + 99.00) = 99.00$$

$$\text{Average value of } y = \frac{1}{3}(2 + 2 + 2) = 2$$

$$\therefore \quad \text{Multiplying constant } \frac{f}{i} = 99.00$$

and $$\text{Additive constant } (f + d) = 2 \text{ m}$$

11.4 ANALLATIC LENS—OBJECT AND THEORY

A. Object

An additional convex lens is provided between the eye-piece and the object glass at a fixed distance from the object glass. This convex lens is known as an anallatic lens. The object of providing the anallatic lens in the telescope is to make the additive constant $(f + d)$ exactly zero.

Such a lens is provided in the external focussing telescope, and is not required in the internal focussing telescope.

The advantage of using this lens is that it simplifies the calculation of distances and heights, as its presence ensures that there is only one constant, namely the multiplying constant. The distances are directly obtained by multiplying the difference of stadia hair readings by 100.

One disadvantage of using the anallatic lens is that the brightness of the image is much reduced due to absorption of light.

B. Theory of Anallatic Lens

Figure 11.5 shows an anallatic lens. The notation used is as follows.

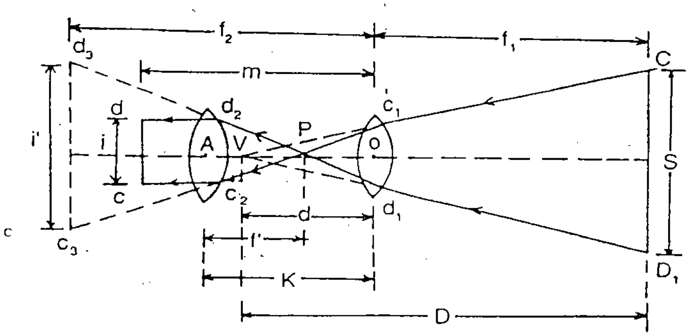


Fig. 11.5

S = staff intercept CD_1
O = optical centre of object glass
A = optical centre of anallatic lens
V = vertical axis of instrument
P = principal focus of the anallatic lens
f_1 and f_2 = conjugate focal lengths of object glass
f = focal length of object glass
f' = focal length of anallatic lens
K = distance between object glass and anallatic lens
d = distance between optical centre of object glass and vertical axis of instrument
m = distance between optical centre of object glass and real image dc
i = length of real image dc, when anallatic lens is provided
i' = length of image d_3c_3 when anallatic lens is not provided
D = distance between vertical axis of instrument and the staff

The rays of light from C and D_1 emanating along CN and DN actually meet at P after refraction while passing through the object glass. The distance (K) is so adjusted that P becomes the principal focus of the anallatic lens. Again the diverging rays from P will emerge in a direction parallel to the axis of the telescope after passing through the anallatic lens. Thus a real image, dc, is formed.

Here, d_3c_3 represents the image if the anallatic lens is not provided.

According to the law of lenses,

$$\frac{1}{f} = \frac{1}{f_1} + \frac{1}{f_2} \qquad (1)$$

and $$\frac{1}{f'} = \frac{1}{(m-k)} - \frac{1}{(f_2 - k)} \qquad (2)$$

where $(m - k)$ and $(f_2 - k)$ together constitute the conjugate focal length of the anallatic lens. In Eq. (2) the sign is negative as d_3c_3 and dc are in the same side of the anallatic lens.

Also,

$$\frac{S}{i'} = \frac{f_1}{f_2} \qquad (3)$$

and $$\frac{i'}{i} = \frac{f_2 - k}{m - k} \qquad (4)$$

By solving these four equations and eliminating m, f_2 and i', we get

$$f_1 = \frac{ff'S}{i(f + f' - k)} - \frac{f(k - f')}{(f + f' - k)}$$

Again,

$$D = f_1 + d$$

$$\therefore \quad D = \frac{ff'}{i(f + f' - k)} \times S - \frac{f(k - f')}{(f + f' - k)} + d \qquad (5)$$

The term $\left\{d - \dfrac{f(k-f')}{(f+f'-k)}\right\}$

should be equal to zero, so that D may be proportional to S.

Thus, $$d - \frac{f(k-f')}{(f+f'-k)} = 0$$

From this, we get

$$k = f' + \frac{fd}{(f+d)} \qquad (6)$$

Now, by adopting suitable values of f, f', k and i, the expression

$$\frac{ff'}{i(f+f'-k)}$$

is made equal to 100, which is the multiplying constant of the instrument.

11.5 METHODS OF TACHEOMETRY

Tacheometry involves mainly two methods:

1. The stadia method
2. The tangential method.

1. The stadia method In this method the diaphragm of the tacheometer is provided with two stadia hairs (upper and lower). Looking through the telescope the stadia hair readings are taken. The difference in these readings gives the staff intercept. To determine the distance between the station and the staff, the staff intercept is multiplied by the stadia constant (i.e. multiplying constant, 100). The stadia method may, in turn, be of two kinds.

(a) The Fixed Hair Method The distance between the stadia hairs is fixed in this method, which is the one commonly used. When the staff is sighted through the telescope, a certain portion of the staff is intercepted by the upper and lower stadia. The value of the staff intercept varies with the distance. However, the distance between the station and the staff can be obtained by multiplying the staff intercept by the stadia constant.

(b) The Moveable Hair Method The stadia hairs are not fixed in this method. They can be moved or adjusted by micrometer screws. The staff is provided with two targets or vanes a known distance apart. During observation, the distance between stadia hairs is so adjusted that the upper hair bisects the upper target and the lower hair bisects the lower target. The variable stadia intercept is measured and the required distance is then computed.

This method is not generally used.

2. The tangential method In this method, the diaphragm of the tacheometer is not provided with stadia hair. The readings are taken by the single horizontal hair. The staff consists of two vanes or targets a known distance apart. To measure the staff intercept, two pointings are required. The angles of elevation or depression are measured and their tangents are used for finding the horizontal distances and elevations.

This method is also not generally used. The stadia method requires only one observation, but the tangential method requires two pointings of the telescope.

11.6 FIXED HAIR METHOD

Case 1 *When line of sight is horizontal and staff is held vertically.*

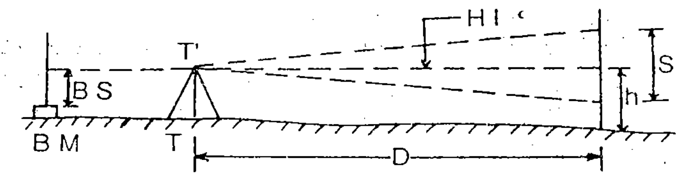


Fig. 11.6

When the line of sight is horizontal, the general tacheometric equation for distance is given by

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

The multiplying constant (f/i) is 100, and additive constant $(f + d)$ is generally zero.

RL of staff station P = HI − h

where HI = RL of BM + BS (HI = height of instrument

h = central hair reading BS = backsight)

Case II *When line of sight is inclined, but staff is held vertically*

Here, the measured angle may be the angle of elevation or that of depression.

(a) Considering Angle of Elevation (Positive)

In Fig. 11.7,

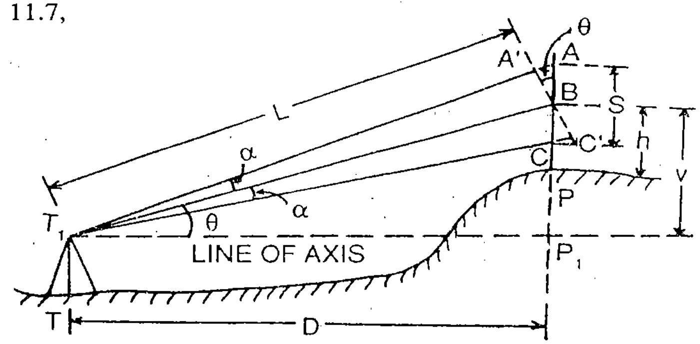


Fig. 11.7

T = instrument station
T_1 = axis of instrument
P = staff station
A, B, C = position of staff cut by hairs
S = AC = staff intercept
h = central hair reading
V = vertical distance between instrument axis and central hair
D = horizontal distance between instrument and staff
L = inclined distance between instrument axis and B
θ = angle of elevation
α = angle made by outer and inner rays with central ray

A′C′ is drawn perpendicular to the central ray, T_1B.

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

Horizontal distance $D = L \cos\theta$.

$$= \frac{f}{i}(A'C')\cos\theta + (f+d)\cos\theta \qquad (1)$$

Now A′C′ is to be expressed in terms of AC (i.e. S).
In Δs ABA′ and CBC′,

$$\angle ABA' = \angle CBC' = \theta$$
$$\angle AA'B = 90^\circ + \alpha$$
$$\angle BC'C = 90 - \alpha$$

The angle α is very small.

$\angle AA'B$ and $\angle BC'C$ may be taken equal to 90°.

So $\qquad AC' = AC\cos\theta = S\cos\theta$

From Eq. (1),

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

$$\therefore \quad D = \frac{f}{i} \times S\cos^2\theta + (f+d)\cos\theta \qquad (1)$$

Again $\qquad V = L\sin\theta$

$$= \left\{\frac{f}{i} \times S\cos\theta + (f+d)\right\}\sin\theta$$

$$= \frac{f}{i} \times S\cos\theta\sin\theta + (f+d)\sin\theta$$

$$V = \frac{f}{i} \times \frac{S \times \sin 2\theta}{2} + (f+d)\sin\theta \qquad (2)$$

Also $\qquad V = D\tan\theta$

RL of staff station P = RL of axis of instrument + $V - h$ (3)

(b) Considering Angle of Depression (negative) In this case also (Fig. 11.8), the expressions for D and V are same as in (a). That is,

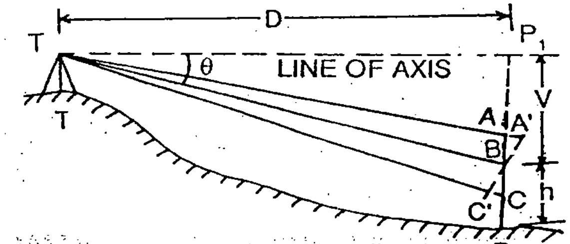


Fig. 11.8

$$D = \frac{f}{i} \times S\cos^2\theta + (f+d)\cos\theta \qquad (4)$$

$$V = \frac{f}{i} \times \frac{S \times \sin^2\theta}{2} + (f+d)\sin\theta \qquad (5)$$

RL of staff station, P = RL of axis of instrument − $V - h$ (6)

Case III Line of sight inclined, but staff normal to it.

(a) Considering Angle of Elevation (Positive) In Fig. 11.9

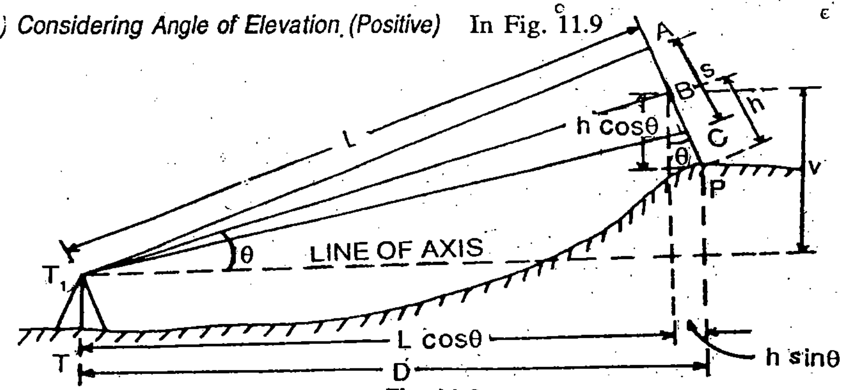


Fig. 11.9

AC = staff intercept (S)
θ = angle of elevation
BP = h (central hair reading)
T_1B = L (inclined distance)

Vertical height of central hair = $h\cos\theta$
Horizontal distance between T and B = $L\cos\theta$
Horizontal distance PP_1 = $h\sin\theta$

Since the staff is perpendicular to the line of collimation,

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

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

$$= \frac{f}{i} \times S \cos \theta + (f + d) \cos \theta + h \sin \theta \qquad (7)$$

Vertical distance $V = L \sin \theta$

$$= \frac{f}{i} \times S \sin \theta + (f + d) \sin \theta \qquad (8)$$

RL of staff station P = RL of instrument axis + $V - h \cos \theta$ (9)

(b) Considering Angle of Depression (Negative) According to Fig. 11.10, horizontal distance,

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

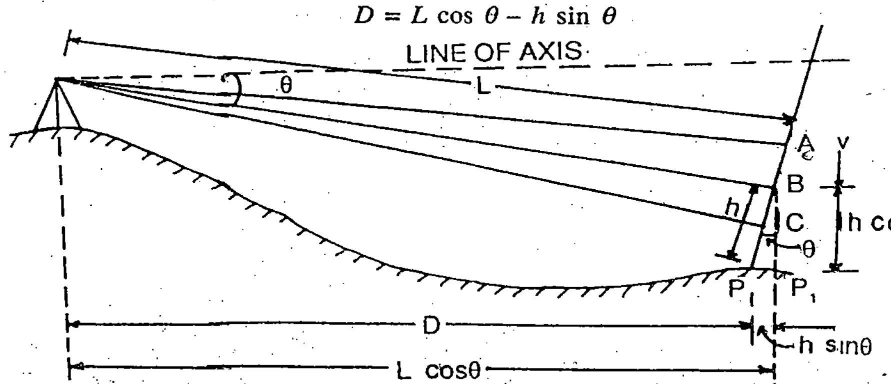


Fig. 11.10

Again, $L = \frac{f}{i} \times S + (f + d)$

$\therefore$ $D = \frac{f}{i} \times S \times \cos \theta + (f + d) \cos \theta - h \sin \theta$ (10)

Vertical distance, $V = L \sin \theta$

$\therefore$ $V = \frac{f}{i} \times S \sin \theta + (f + d) \sin \theta$ (11)

RL of P = RL of instrument axis $- V - h \cos \theta$

11.7 WORKED-OUT PROBLEMS ON FIXED HAIR METHOD OF TACHEOMETRY

Problem 1 A tacheometer was set up at a station C and the following readings were obtained on a staff vertically held.

Inst. station	Staff station	Vertical angle	Hair readings (m)	Remark
C	BM	– 5°20′	1.50, 1.800, 2.450	RL of BM = 750.50 m
C	D	+ 8°12′	0.750, 1.500, 2.250	

Calculate the horizontal distance CD and RL of D, when the constants of instrument are 100 and 0.15.

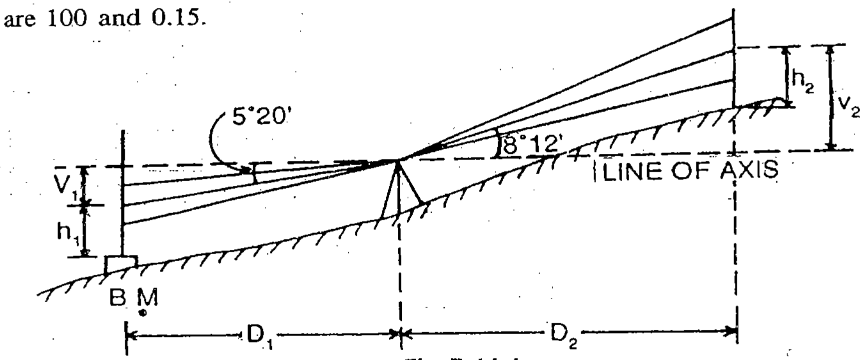


Fig. P.11.1

Solution

When the staff is held vertically, the horizontal and vertical distances are given by the relations

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

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

Here $\frac{f}{i} = 100$ and $(f + d) = 0.15$

In the first observation, $S_1 = 2.450 - 1.150 = 1.300$ m

$\theta_1 = 5°20'$ (depression)

$$V_1 = 100 \times 1.300 \times \frac{\sin 10°40'}{2} + 0.15 \times \sin 5°20' = 12.045 \text{ m}$$

In the second observation, $S_2 = 2.250 - 0.750 = 1.500$

$\theta_2 = 8.12'$ (elevation)

$$V_2 = 100 \times 1.500 \times \frac{\sin 16°24'}{2} + 0.15 \times \sin 8°12' = 21.197 \text{ m}$$

$$D_2 = 100 \times 1.500 \times \cos^2 8°12' + 0.15 \times \cos 8°12' = 147.097 \text{ m}$$

RL of instrument axis = RL of BM + h_1 + V_1

= 750.500 + 1.800 + 12.045 = 764.345 m

$$\text{RL of } D = \text{RL of inst. axis} + V_2 - h_2$$
$$= 764.345 + 21.197 - 1.500 = 784.042 \text{ m}$$

So, the distance CD = 147.097 m and RL of D = 784.042 m.

Problem 2 The following observations were taken with a tacheometer fitted with an anallatic lens, the staff being held vertically. The constant of the tacheometer is 100.

Int. station	Height of instrument	Staff station	Vertical angle	Staff readings (m)	Remark
P	1.255	BM	– 4°20′	1.325, 1.825, 2.325	RL of BM = 255.750 m
P	1.255	A	+ 6°30′	0.850, 1.600, 2.350	
B	1.450	A	– 7°24′	1.715, 2.315, 2.915	

Calculate the RL of B and the distance between A and B.

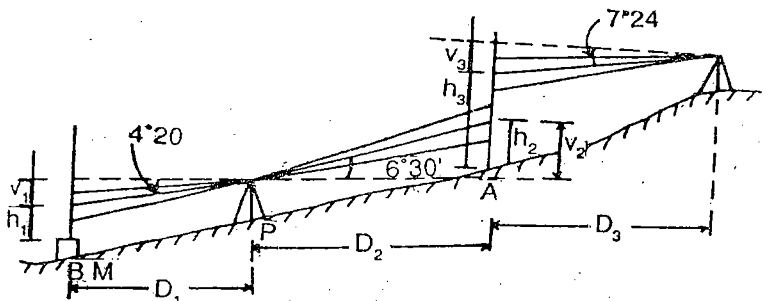


Fig. P.11.2

Solution

Here, Multiplying constant, $f/i = 100$ and Additive constant, $f + d = 0$

Since, the staff is held vertically, the vertical distance is given by

$$V = \frac{f}{i} \times S \times \frac{\sin 2\theta}{2}$$

In the first observation,

$$V_1 = 100\,(2.325 - 1.325) \times \frac{\sin 8°40'}{2} = 7.534 \text{ m}$$

In the second observation,

$$V_2 = 100\,(2.350 - 0.850) \times \frac{\sin 13°0'}{2} = 16.871 \text{ m}$$

In the third observation,

$$V_3 = 100\,(2.915 - 1.715) \times \frac{\sin 14°48'}{2} = 15.326 \text{ m}$$

$$\text{RL of axis when Inst. at P} = \text{RL of BM} + h_1 + V_1$$
$$= 255.750 + 1.825 + 7.534 = 265.109 \text{ m}$$
$$\text{RL of A} = 265.109 + V_2 - h_2$$
$$= 265.109 + 16.871 - 1.600 = 280.380 \text{ m}$$
$$\text{RL of axis when inst. at B} = 280.380 + h_3 + V_3$$
$$= 280.380 + 2.315 + 15.326 = 298.021 \text{ m}$$
$$\text{RL of B} = 298.021 - \text{HI}$$
$$= 298.021 - 1.450 = 296.571 \text{ m}$$
$$\text{Distance between A and B, } D_3 = 100\,(2.915 - 1.715) \times \cos^2 7°24'$$
$$= 118.009 \text{ m}$$

Problem 3 The following observations were made in a tacheometric survey.

Inst. station	Height of axis	Staff station	Vertical angle	Hair readings (m)	Remark
A	1.345	BM	– 5°30′	0.905, 1.455, 2.005	RL of RM = 450.500 m
A	1.345	B	+ 8°0′	0.755, 1.655, 2.555	
B	1.550	C	+ 10°0′	1.500, 2.250, 3.000	

Calculate the RLs of A, B and C, and the horizontal distances AB and BC. The tacheometer is fitted with an anallatic lens and the multiplying constant is 100.

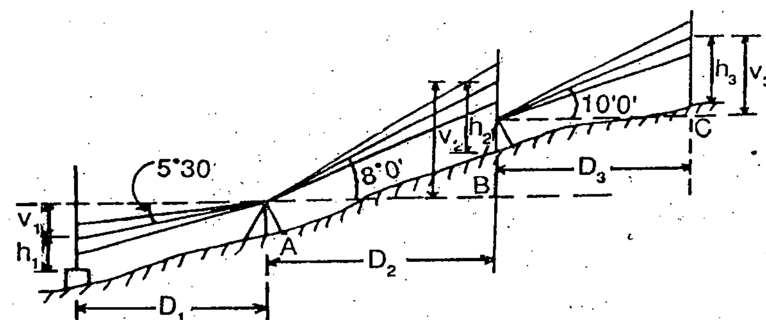


Fig. P. 11.3

Solution

Here $\frac{f}{i} = 100$ and $(f + d) = 0$

Since the staff is held vertically,

$$\text{Horizontal distance } D = \left(\frac{f}{i}\right) \times S \cos^2 \theta$$

$$\text{Vertical distance } V = \left(\frac{f}{i}\right) \times S \times \frac{\sin^2 \theta}{2}$$

In the first observation,

$$V_1 = 100 \times (2.005 - 0.905) \times \frac{\sin 11^\circ}{2} = 10.494 \text{ m}$$

$$D_1 = 100 \times (2.005 - 0.905) \times \cos^2 5^\circ 30' = 108.989 \text{ m}$$

In the second observation,

$$V_2 = 100\,(2.555 - 0.755) \times \frac{\sin 16^\circ}{2} = 24.807 \text{ m}$$

$$D_2 = 100\,(2.555 - 0.755) \times \cos^2 8^\circ = 176.514 \text{ m}$$

In the third observation,

$$V_3 = 100 \times (3.000 - 1.500) \times \frac{\sin 20^\circ}{2} = 25.652 \text{ m}$$

$$D_3 = 100 \times (3.000 - 1.500) \times \cos^2 10^\circ = 145.477 \text{ m}$$

Distance AB = D_2 = 176.514 m

Distance BC = D_3 = 145.477 m

RL of axis when inst. at A = RL of BM + $V_1 + h_1$
= 450.500 + 10.494 + 1.455 = 462.449 m

RL of A = 462.449 height of axis
= 462.449 − 1.345 = 461.104 m

RL of B = 462.449 + $V_2 - h_2$
= 462.449 + 24.807 − 1.655 = 485.601 m

RL of axis when inst. at B = 485.601 + 1.550 = 487.151 m

RL of C = 487.151 + $v_3 - h_3$
= 487.151 + 25.652 − 2.250 = 510.553 m

Problem 4 The following observation were made using a tacheometer fitted with an anallatic lens, the multiplying constant being 100.

Inst. station	Height of inst.	Staff station	WCB	Vertical angle	Hair readings	Remarks
0	1.550	A	30°30′	4°30′	1.155, 1.755 2.355	RL of O = 150.000
		B	75°30′	10°15′	1.250, 2.000, 2.750	

Calculate the distance AB, and the RLs of A and B. Find also the gradient of the line AB.

Solution

According to Fig. P.11.4,

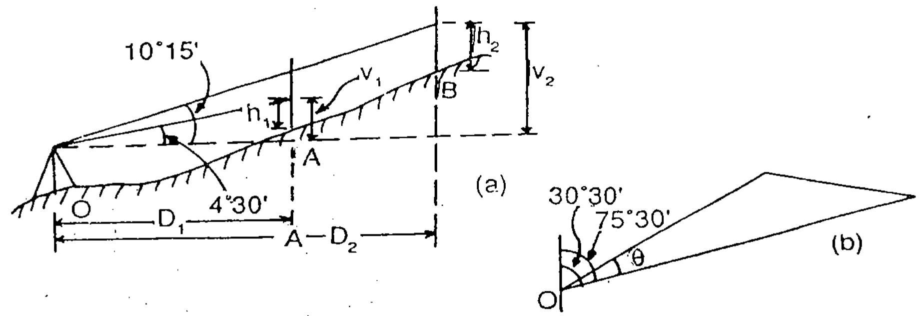


Fig. P.11.4

In the first observation:

$$V_1 = 100 \times (2.355 - 1.155) \times \frac{\sin 9^\circ}{2} = 9.386 \text{ m}$$

$$D_1 = 100 \times (2.355 - 1.155) \times \cos^2 4^\circ 30' = 119.261 \text{ m}$$

In the second observation:

$$V_2 = 100\,(2.750 - 1.250) \times \frac{\sin 20^\circ 30'}{2} = 26.265 \text{ m}$$

$$D_2 = 100\,(2.750 - 1.250) \times \cos^2 10^\circ 15' = 145.250 \text{ m}$$

RL of axis = RL of O + height of inst.
= 150.000 + 1.550 = 151.550 m

RL of A = 151.550 + $V_1 - h_1$
= 151.550 + 9.386 − 1.755 = 159.181 m

RL of B = 151.550 + $V_2 - h_2$
= 151.550 + 26.265 − 2.000 = 175.815 m

OA = D_1 = 119.261 m

OB = D_2 = 145.250 m

θ = 75°30′ − 30°30′ = 45°0′

$$AB = \sqrt{OA^2 + OB^2 - 2 \times OA \times OB \times \cos 45^\circ}$$

$$= \sqrt{(119.261)^2 + (145.250)^2 - 2 \times 119.261 \times 145.250 \times 0.707}$$

= 104.05 m

Difference of level between

A and B = 175.815 − 159.181 = 16.634 m (rise from A to B)

$$\text{Gradient of AB (rising)} = \frac{16.634}{104.05} = \frac{1}{6.25}$$ i.e. 1 in 6.25.

Problem 5 Two points A and B are on opposite sides of a summit. The tacheometer was set up at P on top of the summit, and the following readings were taken.

Inst. station	Height of inst.	Staff station	Vertical angle	Hair readings	Remark
P	1.500	A	– 10°0′	1.150, 2.050, 2.950	RL of P = 450.500 m
P	1.500	B	– 12°0′	0.855, 1.605, 2.355	

The tacheometer is fitted with an anallatic lens, the multiplying constant being 100. The staff was held normal to the line of sight.

Find: (a) The distance between A and B, and
(b) The gradients of lines PA and PB.

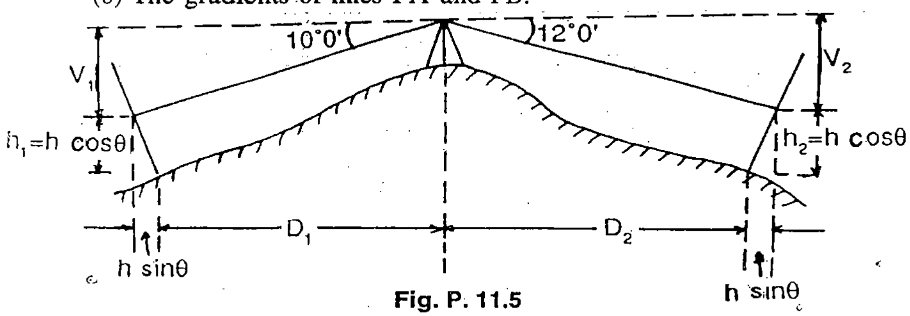


Fig. P. 11.5

Solution

We know that when the staff is held normal to the line of sight, the vertical distance is given by

$$V = \frac{f}{i} \times S \sin\theta + (f + d)\sin\theta$$

Here $\frac{f}{i} = 100$ and $(f + d) = 0$

$\theta_1 = 10°$ and $\theta_2 = 12°$

From Eq. (11)

$$V_1 = \frac{f}{i} \times S \sin\theta_1 = 100 \times (2.950 - 1.150) \times \sin 10° = 31.256 \text{ m}$$

Similarly, $V_2 = 100\ (2.355 - 0.855) \sin 12° = 31.186$ m

$h_1 = 2.050 \times \cos 10° = 2.018$ m

$h_2 = 1.605 \times \cos 12° = 1.569$ m

RL of A inst. axis = 450.500 + 1.500 = 452.000 m

RL of A = RL of inst. axis – V_1 – h_1
= 452.000 – 31.256 – 2.018 = 418.726 m

RL of B = 452.000 – V_2 – h_2
= 452.000 – 31.186 – 1.569 = 419.245 m

The horizontal distances are given by equation,

$$D = \frac{f}{i} \times S \cos\theta + (f + d)\cos\theta - h\sin\theta$$

Hence, $D_1 = 100 \times (2.950 - 1.150) \cos 10° - 2.050 \sin 10°$
$= 177.265 - 0.355 = 176.91$ m

$D_2 = 100\ (2.355 - 0.855) \cos 12° - 1.605$ in $12°$
$= 146.722 - 0.333 = 146.389$ m

Distance between A and B = $D_1 + D_2$
= 176.910 + 146.389 = 323.299 m

$$\text{Gradient of PA (falling)} = \frac{450.500 - 418.726}{176.910} = \frac{1}{5.56} \quad \text{(1 in 5.56)}$$

$$\text{Gradient of PB (falling)} = \frac{450.500 - 419.245}{146.389} = \frac{1}{4.68} \quad \text{(1 in 4.68)}$$

Problem 6 The following are the records of a tacheometric survey:

Inst. Station	Staff station	Bearing	Vertical angle	Hair readings
A	B	N30°30′ E	+ 10°0′	1.250, 1.750, 2.250
B	C	S40°0′ E	+ 5°0′	0.950, 1.750, 2.550
C	D	S45°0′ W	+ 8°0′	1.550, 2.150, 2.750

Multiplying constant = 100, and additive constant = 0. The staff is held vertically. Calculate the length and bearing of DA.

Solution The distances are calculated from the formula

$$D = \frac{f}{i} \times S \cos^2\theta$$

AB = 100 (2.250 – 1.250 – 1.250) × cos² 10° = 96.98 m
BC = 100 (2.500 – 0.950) × cos² 5° = 158.78 m
CD = 100 (2.750 – 1.550) × cos² 8° = 117.67 m

Let Length of DA = L and Bearing of DA = θ

Latitude	*Departure*
AB = + 96.98 cos 30°30′	AB = + 96.98 sin 30°30′
= + 83.40 (northing)	= + 49.22 (easting)
BC = – 158.78 cos 40°0′	BC = + 158.78 sin 40′0′
= – 121.63 (southing)	= + 102.06 (easting)
CD = – 117.67 cos 45°0′	CD = – 117.67 sin 45°0′
= – 83.20 (southing)	= – 83.20 (westing)
DA = $L \cos\theta$	DA = $L \sin\theta$

For a closed traverse, the algebraic sum of latitude and departures must equal to zero.

So, $+83.40 - 121.63 - 83.20 + 1 \cos\theta = 0$

or $L\cos\theta = 121.43$ (1)

and $+49.22 + 102.06 - 83.20 + 1 \sin\theta = 0$

$L\sin\theta = -68.08$ (2)

Since the latitude is positive and departure is negative, the line DA lies in the NW quadrant.

$$\tan\theta = \frac{68.08}{121.43} = 0.5605$$

$$\theta = 29°16'38''$$

$\therefore$ Bearing of DA = N 29°16′38″ W

$$\text{Length DA} = \sqrt{(121.43)^2 + (68.08)^2} = 139.21 \text{ m}$$

Problem 7 The following observations were taken from traverse stations A and B to points C and D by means of a stadia techeometer fitted with an anallatic lens, the instrument constant being 100.

Inst. station	Staff station	Height of inst.	Bearing	Vertical angle	Staff reading
A	C	1.48	126°30′	+ 12°10′	0.77, 1.60, 2.43
B	D	1.42	184°45′	– 10°30′	0.86, 1.84, 2.82

Coordinates of A = 112.8 N, 106.4 W
Coordinates of B = 198.5 N, 292.6 W
Determine the length of the line CD.

Solution

Distance AC = $100 \times (2.43 - 0.77) \times \cos^2 12°10' = 158.63$ m
Distance BD = $100\,(2.82 - 0.86) \times \cos^2 10°30' = 189.49$ m
Reduced bearing of AC = S53°30′ E
Reduced bearing of BD = S4°45′ W

Line	Latitude	Departure
AC	– 158.63 cos 53°30′ = – 94.36 m	+ 158.63 sin 53°30′ = + 127.52 m
BD	– 189.49 cos 4°45′ = – 188.84 m	– 189.49 sin 4°45′ = – 15.69 m

Coordinates of C

Latitude of A = + 112.82 m (northing)
Total latitude of C = + 112.82 – 94.36 = 18.44 m

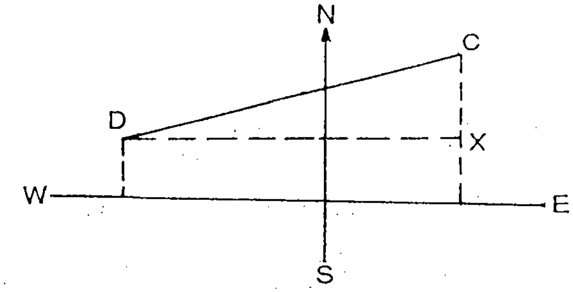


Fig. P.11.6

Departure of A = – 106.4 m (westing)
Total departure of C = – 106.40 + 127.52 = + 21.12 m

Coordinates of D

Latitude of B = + 198.5 m (northing)
Total latitude of D = + 198.50 – 188.84 = + 9.66 m
Departure of B = – 292.6 m (westing)
Total departure of D = – 292.6 – 15.69 = 308.29 m

DX = departure of C + departure of D
= 21.12 + 308.29 = 329.41 m

CX = latitude of C – latitude of D
= 18.44 – 9.66 = 8.78 m

$$\text{Length CD} = \sqrt{(DX)^2 + (CX)^2}$$

$$= \sqrt{(329.4)^2 + (8.78)^2} = 329.52 \text{ m}$$

11.8 THE MOVEABLE HAIR METHOD

In this method the staff intercept is kept constant, but the distance between the stadia wires is variable. The staff is provided with two targets a known distance apart and a third target in the middle. The theodolite is provided with a special type of diaphragm shown in Fig. 11.11. This type of theodolite is known as subtense theodolite. The diaphragm consists of a central wire fixed with the axis of the telescope. The upper and lower stadia wires can be moved by micrometer screws in a vertical plane. The distance by which the stadia wires are moved is measured according to the number of turns of the micrometer screws. The number of complete turns is read on the scale, and the fractional parts are read on the drum of the micrometer screws provided one above and one below the eye-piece. The sum of the micrometer readings is taken in order to obtain the total distance moved by the stadia wires.

For taking the observation, the middle target is first bisected by the central wire. Then the micrometer screws are simultaneously turned to move the stadia wires until the upper and lower targets are bisected.

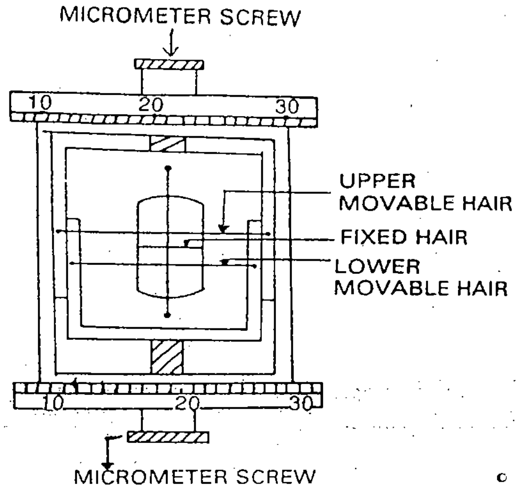


Fig. 11.11

When the line of sight is horizontal, the distance is given by

$$D = \frac{C \times S}{n} + (f + d)$$

where C = constant varying from 600 to 1,000
n = sum of the readings in the micrometer
S = staff intercept (distance between targets)

When the line of sight is inclined, the distance is calculated by a formula similar to the one used for the fixed hair method, namely

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

Example 1 The micrometer readings of a subtense theodolite are 3.455 and 3.405. The distance between the targets is 3 m. The constants of the instrument are 600 and 0.5 m. Calculate the distance between the instrument and the staff.

Solution

Here $C = 600 \qquad (f + d) = 0.5$

$n = 3.455 + 3.405 = 6.860$

$S = 3$ m

$$\text{Distance} = \frac{C \times S}{n} + (f + d)$$

$$= \frac{600 \times 3}{6.860} + 0.5 = 262.89 \text{ m}$$

Example 2 The same distance was measured by a tacheometer and a subtense theodolite.

Records of the tacheometer reading are as follows:

Staff intercept = 1.255 m, Angle of elevation = 5°0′

$$\frac{f}{i} = 100 \qquad (f + d) = 0.2$$

Subtense theodolite readings:

Staff intercept = 2 m Angle of elevation = 5°30′
Constants = 1000 and 0.3

Find the total number of turns in micrometer.

Solution

In the first case $D = \frac{f}{i} \times S \cos^2 \theta + (f + d) \cos \theta$

$$= 100 \times 1.255 \times \cos^2 5° + 0.2 \cos 5° = 124.5 + 0.20$$

$$= 124.75 \text{ m} \qquad (1)$$

In the second case $n = ?,\ C = 1{,}000,\ S = 2$ m, $(f + d) = 0.3$
$\theta = 5°30'$

$$D = \frac{C \times S \times \cos^2 5°30'}{n} + (f + d) \cos 5°30'$$

$$= \frac{1{,}000 \times 2 \times 0.99}{n} + 0.3 \times 0.9953$$

$$= \frac{1{,}980}{n} + 0.2985 \qquad (2)$$

From Eqs (1) and (2), $\frac{1{,}980}{n} + 0.2985 = 124.75$

or $\frac{1{,}980}{n} = 124.45 \qquad n = \frac{1{,}980}{124.45} = 15.91$

11.9 THE TANGENTIAL METHOD OF TACHEOMETRY

This method is used when the theodolite is a simple transit type and does not carry a stadia diaphragm. The staff consists of two vanes or targets a known distance apart. The angles of elevation or depression of the targets are measured by theodolite. The horizontal and vertical distances are computed as explained below.

Case I —When both angles of target are angles of elevation.

In this case, the staff is held vertically. In Fig. 11.12,

T = instrument station
T_1 = instrument axis
A = staff station
V = vertical distance between lower vane and axis of instrument
S = distance between targets
θ_1 = vertical angle made by upper target
θ_2 = vertical angle made by lower target
h = height of lower vane above the staff station

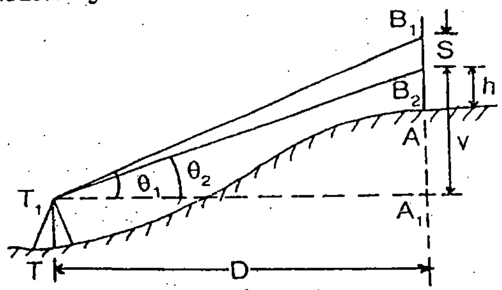


Fig. 11.12

From the figure, $V + S = D \tan \theta_1$

and $V = D \tan \theta_2$

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

or $$D = \frac{S}{(\tan \theta_1 - \tan \theta_2)} \quad (1)$$

$\therefore$ $$V = \frac{S \tan \theta_2}{(\tan \theta_1 - \tan \theta_2)} \quad (2)$$

RL of A = RL of instrument axis + $V - h$

Case II—When both angles of target are angles of depression

In such a situation, the staff is held vertically.

The notation used is the same as in Case I.

In Fig. 11.13,

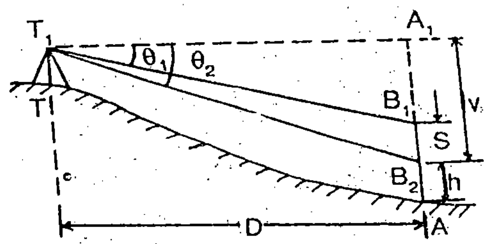


Fig. 11.13

$V = D \tan \theta_2$

and $V - S = D \tan \theta_1$

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

or $$D = \frac{S}{(\tan \theta_2 - \tan \theta_1)} \quad (3)$$

$\therefore$ $$V = D \tan \theta_2 = \frac{S \tan \theta_2}{(\tan \theta_2 - \tan \theta_1)} \quad (4)$$

RL of A = RL of instrument axis $- V - h$

Case III—When one angle is that of elevation and the other that of depression

The staff is held vertically.

Here θ_1 is indicated by a positive sign (angle of elevation) and θ_2 is by a negative sign (angle of depression).

The notations used in the same as in Case I.

In Fig. 11.14,

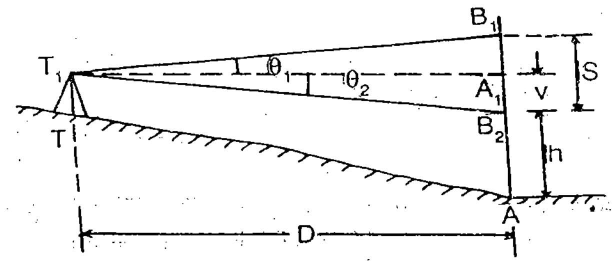


Fig. 11.14

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

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

$\therefore$ $$D = \frac{S}{(\tan \theta_1 + \tan \theta_2)} \quad (5)$$

and $$V = D \tan \theta_2 = \frac{S \tan \theta_2}{(\tan \theta_1 + \tan \theta_2)} \quad (6)$$

RL of A = RL of instrument axis $- V - h$

The only advantage of the moveable hair method is that the survey can be conducted by ordinary transit theodolite, which does not carry a stadia diaphragm.

The disadvantages are as follows:

1. Two vertical angles are to be measured in each observation; hence this method requires comparatively more time.
2. During the interval between the measurement of angles, the theodolite may be disturbed; this may cause wastage of time.
3. This method is very tedious.

Notes: (1) The most common method used nowadays is the fixed hair method with the staff held vertically.
(2) The moveable hair and tangential methods are not generally adopted.

Example 1 The following observations were taken with a transit theodolite.

Inst. station	Staff station	Target	Vertical angle	Staff reading	Remark
O	A	Lower	+ 4°30′	0.950	RL of the instrument
		Upper	+ 6°30′	3.250	axis = 255.500

Calculate the horizontal distance between the instrument station and staff, as also the RL of staff station A.

Solution

In Fig. E.11.1,

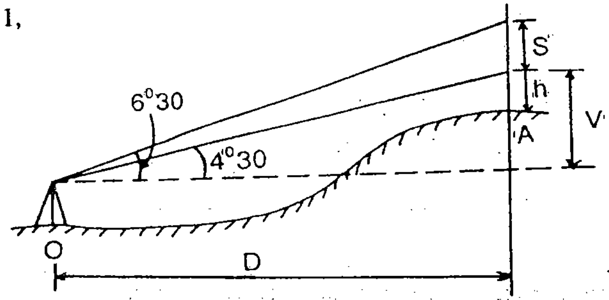


Fig. E.11.1

$$S = 3.250 - 0.950 = 2.300 \qquad h = 0.950$$

$$S + V = D \tan 6°30' \qquad (1)$$

Again $\quad V = D \tan 4°30' \qquad (2)$

From Eqs (1) and (2)

$$S = D\,(\tan 6°30' - \tan 4°30')$$

$$D = \frac{S}{(\tan 6°30' - \tan 4°30')}$$

$$= \frac{2.300}{(\tan 6°30' - \tan 4°30')} = 65.340 \text{ m}$$

From Eq. (2),

$$V = 65.340 \times \tan 4°30' = 5.142 \text{ m}$$

$$\text{RL of A} = \text{RL of instrument axis} + V - h$$

$$= 255.500 + 5.142 - 0.950 = 259.692 \text{ m}$$

Example 2 To find the RL of station B, two observations are taken by a theodolite from station A—one to a BM and the other to the station B. The records are as follows:

Inst. station	Staff station	Target	Vertical angle	Staff reading	Remark
A	BM	Lower	– 10°0′	0°655	RL of BM
		Upper	– 7°0′	2.655	= 510.500 m
A	B	Lower	– 5°0′	1.250	
		Upper	+ 4°0′	3.200	

Find the RL of B, and the distance between the BM and station B.

Solution Consider Fig. E.11.2.

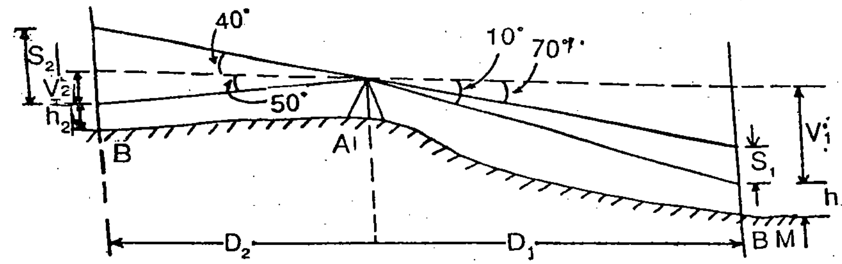


Fig. E.11.2

First observation $\quad S_1 = 2.655 - 0.655 = 2.000$ m

$$h_1 = 0.655 \text{ m} \qquad V_1 = D_1 \tan 10°$$

$$V_1 - S_1 = D_1 \tan 7°$$

$$S_1 = D_1\,(\tan 10° - \tan 7°)$$

$$D_1 = \frac{2.000}{(\tan 10° - \tan 7°)} = 37.31 \text{ m}$$

$$V_1 = 37.31 \times \tan 10° = 6.578 \text{ m}$$

$$\text{RL of instrument axis} = \text{RL of BM} + h_1 + V_1$$

$$= 510.500 + 0.655 + 6.578$$

$$= 517.733 \text{ m}$$

Second observation

$$V_2 = D_2 \tan 5° \qquad S_2 = 3.200 - 1.250 = 1.950 \text{ m}$$

$$S_2 = V_2 = D_2 \tan 4° \qquad h_2 = 1.250 \text{ m}$$

$$S_2 = D_2\,(\tan 5° + \tan 4°)$$

$$D_2 = \frac{1.950}{(\tan 5° + \tan 4°)} = 12.396 \text{ m}$$

$$V_2 = D_2 \tan 5° = 12.396 \times \tan 5° = 1.085 \text{ m}$$

$$\text{RL of B} = \text{RL of instrument axis} - V_2 - h_2$$

$$= 517.733 - 1.085 - 1.250$$

$$= 515.398 \text{ m}$$

Distance between BM and B = 37.31 + 12.396 = 49.706 m $\quad (D_1 + D_2)$

11.10 REDUCTION OF READINGS

The calculation of horizontal and vertical distances by direct application of formulae every time is very laborious. So, for easy computation of readings, some methods of reduction are employed in practice. These methods involve the use of:

1. Tacheometric tables
2. Reduction diagrams

1. Tacheometric tables These tables are prepared for a 1 m staff intercept when the staff is held vertically and constants are 100 and 0.

It is known from tacheometric formulae that:

$$\text{Horizontal distance } D = \frac{f}{i} \times S\cos^2\theta \qquad \left(\frac{f}{i} = 100, S = 1\right)$$

$$= 100\cos^2\theta$$

$$\text{Vertical distance } V = \frac{f}{i} \times S \times \frac{\sin 2\theta}{2} = 50 \times \sin 2\theta$$

Substituting the values of θ from 0° to 20°, a table is prepared for horizontal and vertical distances. The staff intercept and vertical angles are noted. Then the corresponding values of horizontal and vertical distance can be obtained directly from the table.

2. Reduction diagrams Graphs which are prepared for measuring corrections for horizontal distances and for measuring vertical distances directly, are known as reduction diagrams.

(a) Reduction Diagram for Horizontal Correction We know that for horizontal sight

$$\text{Horizontal distance } D = \frac{f}{i} \times S$$

and that for inclined sight,

$$\text{Horizontal distance, } D' = \frac{f}{i} \times S\cos^2\theta$$

$$\text{Horizontal correction, } D - D' = \frac{f}{i} \times S(1 - \cos^2\theta) = \frac{f}{i} \times S\sin^2\theta$$

$$= 200\sin^2\theta \qquad \left(\text{taking } \frac{f}{i} = 100 \text{ and } S = 2\right)$$

The values of $200\sin^2\theta$ are calculated with θ varying from 0 to 10° (taking an interval 30′) and the results are plotted as shown in Fig. 11.15.

The horizontal correction for the horizontal distance 150 m at angle of 6° is marked by P. The amount of correction can be obtained from the right-hand side BC. The correction is always subtracted to get the correct value.

Suppose Multiplying constant = 100
Staff intercept = 1.50
Vertical angle = 6°

Then, Computed horizontal distance = 1.50 × 100 = 150 m
Horizontal correction for 6° = 4.50 m
True horizontal distance = 150 − 4.50 = 145.50 m

(b) Reduction Diagram for Vertical Distance We know that vertical distances are given by

$$V = \frac{f}{i} \times S \times \frac{\sin 2\theta}{2} \qquad \left(\text{taking } \frac{f}{i} = 100 \text{ and } S = 2\right)$$

$$= 100\sin 2\theta$$

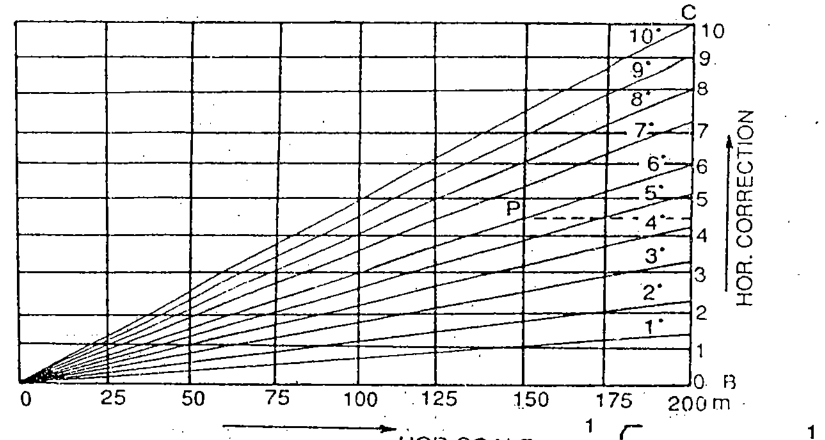


Fig. 11.15

The values of $100\sin 2\theta$ are calculated for different values of θ from 0 to 10° with intervals of 10′ up to 5°. The intervals are generally increased above 5°. The results are plotted as shown in Fig. 11.16.

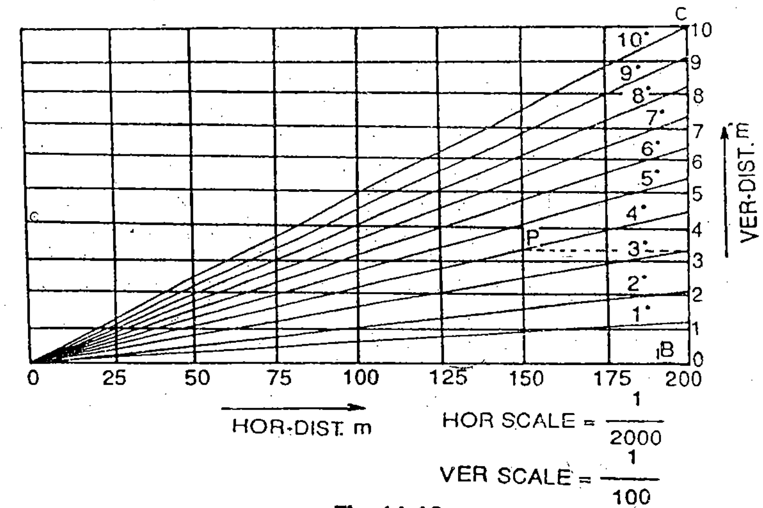


Fig. 11.16

Suppose Multiplying constant = 100
Staff intercept = 1.50
Vertical angle = 4°

Then, Computed horizontal distance = 1.50 × 100 = 150 m

The vertical distance corresponding to 150 m is marked by p. The required vertical distance is noted from the right-hand side BC, which is 3.50 m.

11.11 DIRECT READING OR AUTOREDUCTION TACHEOMETER

In tacheometric survey, the computation involved is very tedious and requires much time. To simplify computation, some direct-reading or autoreduction tacheometers have been designed. These instruments give the horizontal distances and vertical distances directly from the staff reading without the need for measuring vertical angles. There are three types of such instruments:

1. The Jeff cott direct reading tacheometer
2. The Szepessy direct reading tacheometer
3. The Hammer-Fennel autoreduction tacheometer

1. The Jeff cott direct reading tacheometer Such a tacheometer is shown in Fig. 11.17.

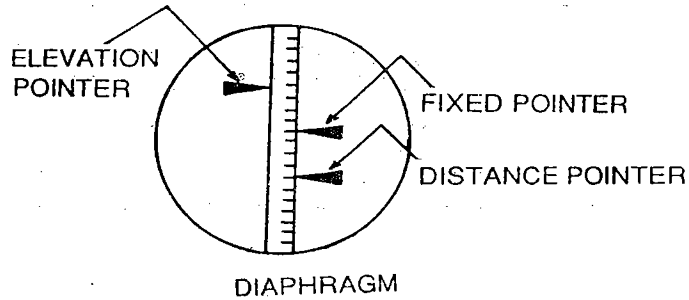


Fig. 11.17

The diaphragm of this instrument carries three pointers—the middle one is fixed and the other two are moveable. The right-hand moveable pointer is known as the distance pointer and the left-hand pointer is called the elevation pointer. The moveable pointers are fitted with cam and lever arrangements by which they are automatically set at the staff reading when the telescope is raised or lowered. In this instrument the constant for horizontal distance is 100 and that for vertical distance is 10. To obtain the horizontal distance, the staff intercept between the fixed and distance pointer is multiplied by 100. To get the vertical height, the staff intercept between the fixed pointer and elevation pointer is multiplied by 10.

Example Suppose the staff readings are 2.350, 1.150 and 0.750. Then,

$$\text{Horizontal distance} = (2.350 - 1.150) \times 100 = 120 \text{ m}$$
$$\text{Vertical distance} = (1.150 - 0.750) \times 10 = 4 \text{ m}$$

2. The Szepessy direct reading tacheometer This instrument consists of a scale of tangents of vertical angles engraved in a glass arc. The scale is fixed with the vertical circle cover. It is brought into view by a prism. When the staff is sighted, the images of the staff and the scale are seen simultaneously. The scale is graduated to a minimum value of, 0.005, and is numbered at every 0.01 as 1, 2, 3, ...

This means the graduation numbered 25 indicates the angle whose tangent is $0.01 \times 25 = 0.25$.

While taking observations, the horizontal crosshair is set at some numbered

division of the scale. Then the staff reading corresponding to this division is noted.

Now the staff intercept between the minimum graduation (i.e. 0.005) immediately above and below the numbered division is noted.

To get horizontal distance, this intercept is multiplied by 100. To obtain vertical distance, this intercept is multiplied by the number at which the cross-hair was fixed.

Example Suppose the cross hair is fixed at number 20 and the staff intercept is 0.65.

Then,
$$\text{Horizontal distance} = 0.65 \times 100 = 65 \text{ m}$$
$$\text{Vertical distance} = 0.65 \times 20 = 13.0 \text{ m}$$

3. Hammer-Fennel autoreduction tacheometer This tacheometer consists of four curves which are seen in the field of view when focussing is done towards the staff. The curves are designated by letters N, E, d and D, where:

(a) The curve N is known as the zero curve.
(b) The curve E is known as the distance curve. The multiplying constant of this curve is 100.
(c) d is known as the height curve. It is used to measure angles of up to ± 47° (a positive sign indicates elevation, and a negative sign a depression). Its multiplying constant is 20.
(d) D is also known as height curve. It is used for measuring angles of up to ± 14° (a positive sign is used for elevation, and a negative sign for depression). The multiplying constant of this curve is 10.

For taking the observation, the zero curve is so adjusted that it is bisected by the zero mark of the staff. The staff readings are then taken with the distance curve (E) and height curve (d or D).

The reading on the distance curve is multiplied by 100 to get horizontal distance, and that on the height curve (it may be d or D) is multiplied by the corresponding constant to obtain vertical distance.

11.12 MEASUREMENT OF HORIZONTAL DISTANCE BY SUBTENSE BAR

The subtense bar is an instrument used for measuring the horizontal distance between the instrument station and a point where the subtense bar is to be set up. In this method a staff or target rod is not necessary, and the theodolite required is also of the ordinary transit type. The targets are bisected by the theodolite and the horizontal angle made by the targets with the instrument station is measured. After this, the horizontal distance is computed. (This is explained later.)

The subtense bar is a metal bar of length varying from 3 to 4 m. There are two discs of diameter about 20 cm at both ends of the bar. The discs are painted black or red in front and white on the other side. In the front side of the disc, a circle of about 7.5 cm diameter is painted in black or white in the central portion. An alidade is provided at the centre of the bar. The alidade is made perpendicular to the axis of the bar. The purpose of providing the alidade is to make the line of sight perpendicular to the axis of the bar. A spirit level is included for levelling. The bar is mounted on a tripod stand which contains a ball-and-socket arrangement for facility of levelling. The discs are generally placed 3 m apart and serve the purpose of targets.

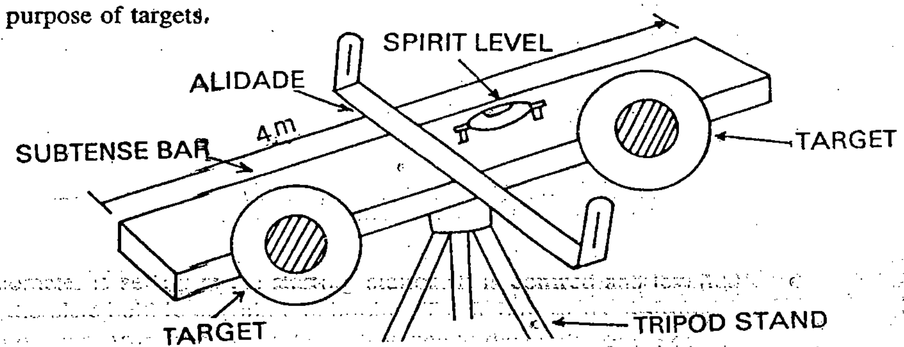


Fig. 11.18

Procedure of observation

1. Consider Fig. 11.19. The transit theodolite is set up at T. The subtense bar is centred and levelled at C, which is the position of the alidade. The points B and D are the discs (targets) 3 m apart. H is the required horizontal distance.

2. With the help of the alidade the line of sight of the telescope of the theodolite is made perpendicular to the axis of the bar.

3. The horizontal angle ∠BTD is measured by the repetition method. Let this angle be θ.

4. Now, TC is perpendicular to BD and C is the middle point of BD.

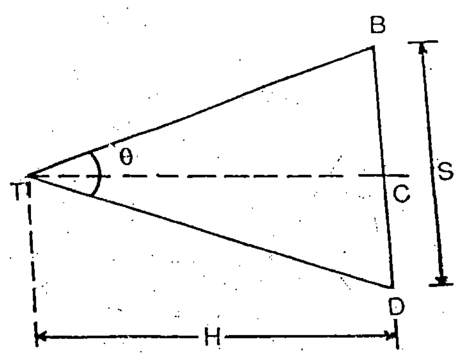


Fig. 11.19

From qBTC,

$$\tan\frac{\theta}{2} = \frac{S/2}{H}$$

or $$H\tan\frac{\theta}{2} = \frac{S}{2} \quad \text{or} \quad H = \frac{1}{2}\left(S\cot\frac{\theta}{2}\right)$$

Thus the horizontal distance can be computed.

11.13 FIELD WORK IN TACHEOMETRY

A tacheometric survey is conducted mainly for preparing a contour map and for

filling interior details in undulating areas where direct chaining is very difficult. By means of a tacheometer the relative distances and RLs of different points can be computed from the instrument station. Such survey is suitable for alignment of highways or railways or for preparing the contour map of a reservoir.

Procedure

1. Reconnaissance: Before starting the work the area to be surveyed is thoroughly examined and the instrument stations are selected according to the nature of the area. If the survey is conducted along a narrow belt, the stations are selected along the centre line of the belt. If the area is comparatively broad, it is to be enclosed by a closed traverse. If the area is very extensive, it is divided into a number of well-conditioned triangles. It should be remembered that every instrument station should command a wide area.

2. The tacheometer is set up at the starting station. It is centred and levelled with respect to the plate bubble and altitude bubble. The height of the instrument (i.e. the vertical distance from the top of the station peg to the centre of the object glass) is measured by levelling staff or stadia rod, looking through the object glass or by any suitable means.

3. The instrument is oriented with reference to any pre-determined station by taking its magnetic bearing, or by any other means.

4. A back sight reading is taken on a nearby benchmark, if available. If not, fly levelling should be done to connect the distant benchmark to the instrument station in order to know the RL of the starting station.

5. The nearby objects or points are demarcated. The magnetic bearings of all of them are measured and noted. The vertical angles are measured with respect to the central horizontal hair. Then three readings on the staff are taken—one for collimation hair and the other two for the upper and lower stadia. These observations are known as side shots. The readings are entered in a tacheometric field book as shown below.

Field Book

Inst. station	Height of station	Staff station	WCB	Vertical angle	Hair readings			Remarks
					Top	Centre	Bottom	

6. Then the tacheometer is directed to the second station, and its magnetic bearing is measured. The vertical angle and three readings on the staff are noted and recorded in the field book.

7. Similarly all the traverse stations are connected and the necessary observations for all the objects are taken from each station and recorded clearly in the field book.

8. From the field book, the distances of the objects from the instrument stations are computed, and the respective RLs of all the points are calculated. A separate table is prepared for the purpose of plotting.

9. The points are plotted on the map according to any suitable scale, and RLs of the respective points are noted by taking the distances and RLs from the prepared table. Then the contour lines may be drawn by the method of interpolation.

Table for Plotting

Inst. station	Staff station	WCB	Horizontal distance	Vertical distance	RL		Remark
					Inst. axis	Staff station	

11.14 ERRORS AND PRECISIONS IN STADIA TACHEOMETRY

A. Errors

The following are the different sources of error in tacheometry.

1. Instrumental error

(a) The adjustment of the tacheometer may not be perfect. To guard against this condition, the permanent adjustment of the tacheometer should be checked, and corrective action taken if necessary.

(b) The graduations of the staff or stadia rod may not be uniform. To eliminate errors due to this condition, either the imperfect staff or rod should be replaced, or necessary corrections should be applied.

(c) The value of the multiplying constant may not be correct. To meet such a situation, the multiplying constant of the instrument should be found out by careful field observation before commencement of the work.

2. Errors of observation

(a) Errors may be caused by incorrect centring and levelling. To prevent such error, the observer should ensure perfect centring, and levelling of the plate bubble and altitude bubble.

(b) The staff may not be held vertically. To avoid such a situation, the verticality of the staff should be verified by plumb bob.

(c) Imperfect focussing of the telescope causes parallax and may lead to incorrect staff readings. To minimise such error, the parallax should always be eliminated by proper focussing.

(d) The sight distance may be much greater than the range of the telescope. To avoid this condition, the sight distance should be such that the graduations of the staff can be seen very distinctly.

3. Errors due to natural causes

(a) During high winds, incorrect staff observations may be taken owing to vibration of instruments. To avoid such error, work should be suspended in extremely windy conditions.

(b) On a very hot day, the different parts of the tacheometer may expand, affecting staff readings. To avoid such error, the instrument should be placed under the shade of a tree or protected by an umbrella.

(c) Due to the glare of the sun, the visibility of the target or staff may be badly affected. To avoid this source of error, the instrument should be so placed that the sun does not reflect from the object glass.

B. Precision in Tacheometry

1. The permissible error in a single horizontal distance should be within 1 in 500 and that in a single vertical distance should not exceed 0.1 m.
2. In an open traverse, average error in distance should not exceed 1 in 850.
3. In a closed traverse, the permissible error of closure in elevation should be $0.08D$ to $0.25D$, where D is the distance in kilometres.
4. In a closed traverse, the permissible closing error should not exceed 0.055 P where P is the perimeter of the traverse.

SHORT QUESTIONS WITH ANSWERS FOR VIVA

Q. 1 What is tacheometry?

Ans. Tacheometry is the branch of surveying in which horizontal and vertical distances are measured by taking angular observations with a tacheometer. The chaining operation is completely eliminated.

Q. 2 What is the difference between a theodolite and tacheometer?

Ans. When a theodolite is fitted with an anallatic lens, it is known as a tacheometer. Without the anallatic lens, the instrument would be called a simple transit theodolite.

Q. 3 Why is an anallatic lens provided in a tacheometer?

Ans. The anallatic lens is provided in a tacheometer to make the additive constant zero.

Q. 4 What is the difference between a fixed hair tacheometer and a subtense theodolite?

Ans. A tacheometer in which the stadia hairs are fixed in a diaphragm, and maintain a constant distance from the central hair is known as a fixed hair tacheometer. But a tacheometer which contains a special diaphragm where the stadia hairs can be moved by micrometer screws in a vertical plane is known as a subtense theodolite.

Q. 5 What are the multiplying constant and additive constant of a tacheometer?

Ans. According to the theory of stadia tacheometry,

$$\text{Horizontal distance } D = \left(\frac{f}{i}\right) \times S + (f + d)$$

the quantity f/i is known as the multiplying constant, and has a value of 100, and the quantity $(f + d)$ is known as additive constant and is equal to zero.

Q. 6 What is a subtense bar?

Ans. It is a simple instrument consisting of a horizontal bar of length about 4 m having two circular discs at the two ends generally 3 m apart. It is used only for measuring the horizontal angle between targets, from which the horizontal distance between the instrument station and the subtense bar may be calculated.

Q. 7 What is tangential tacheometry?

Ans. In this system of tacheometry, the angles of elevation or depression are measured and the distances are calculated from the values of tangents of the angles.

Q. 8 What is the principle of tacheometry?

Ans. The principle of tacheometry is based on the property of isosceles triangles where the distances of the bases from the apices and the lengths of the bases always bear a constant ratio.

Q. 9 What does 'reduction of readings' mean?

Ans. The calculation of horizontal and vertical distances by the direct application of formulae every time is very laborious. So, to simplify calculation, some tables or graphs are prepared which are known as reduction of readings.

Q. 10 What is the purpose of a direct reading tacheometer?

Ans. Direct reading tacheometers are designed to obtain horizontal and vertical distances directly from staff readings without any need for laborious calculations.

Q. 11 What are the various types of direct reading tacheometers?

Ans. The common direct reading tacheometers are:

(a) The Jeff cott direct reading tacheometer,

(b) The Szepessy direct reading tacheometer, and

(c) The Hammer-Fennel auto-reduction tacheometer.

EXERCISES

1. Discuss the methods of tacheometry.
2. Explain the theory of stadia tacheometry.
3. Describe the method of determining the constants of a tacheometer from field measurements.
4. Derive the expressions for horizontal and vertical distances in the fixed hair method when the staff is held vertically and the measured angle is that of elevation.
5. Derive the expressions for horizontal and vertical distances in the fixed hair method when the staff is held normal to the line of sight and the measured angle is that of elevation.
6. Derive the expressions for horizontal and vertical distances by the tangential method when both the angles measured are those of depression.
7. Explain the object and theory of the anallatic lens.
8. Describe with a neat sketch the construction and working of the subtense bar.
9. Describe how tacheometric surveying is conducted in the field.
10. What are the sources of error in tacheometry? What are the permissible errors?
11. Describe the direct reading tacheometers commonly used.
12. What do you mean by reduction of readings? How the reductions are done? (Art. 11.10)
13. A tacheometer fitted with an anallatic lens and having a multiplying constant of 100 was set up at R which is an intermediate point on a traverse leg AB. The following readings were taken with the staff held vertically.

Staff station	Bearing	Vertical angle	Intercept	Axial hair reading
A	40°35′	– 4°24′	2.21	1.99
B	22°35′	– 5°12′	2:02	1.90

Calculate the length AB and the level difference between A and B.

(AMIE, Summer 1989)

(Ans. AB = 68.43 m,
Level difference = 1.24 m)

14. The following observations were taken using a tacheometer fitted with an anallatic lens, the staff being held vertically.

Inst. station	Height of axis	Staff station	Vertical angle	Hair readings	Remarks
P	1.45	BM	– 6°12′	0.98, 1.54, 2.100	RL of BM = 384.25 m
P	1.45	Q	+ 7°5′	0.83, 1.36, 1.89	
Q	1.57	R	+ 12°21′	1.89, 2.48, 3.07	

Determine the distances PQ and QR, and the RLs of P, Q, and R.

(AMIE, Winter 1988)

(Ans. PQ = 104.39 m, QR = 112.60 m,
RL of P = 396.37 m, RL of Q = 409.43 m,
RL of R = 433.17 m)

15. A fixed hair tacheometer fitted with an anallatic lens with an instrument constant of 100, was used to determine the slope between two points P and Q. The following readings were taken. If the staff was held vertically, compute the gradient from P to Q.

Inst. station	Staff station	Bearing	Readings of stadia hair	Reading of axial hair	Vertical angle
A	P	345°	0.915 2.585	1.750	+ 15°
	Q	75°	0.760 3.715	2.240	+ 10°

(AMIE, Summer, 1988)

(Ans. Rising gradient 1 in 39.35)

16. In the course of a tacheometric survey the following observations were made for the normal cross-section of a stream, the instrument being set up on the bank with the telescope level. Assume a multiplying constant of 100.

Staff point	Readings			Remark
	Lower	Central	Upper	
1	4.62	4.76	4.90	Edge of water
2	6.06	6.27	6.48	
3	7.25	7.63	8.01	
4	8.30	8.82	9.34	
5	8.36	8.96	9.57	
6	7.31	8.05	8.80	
7	5.67	6.54	7.41	
8	3.69	4.76	5.81	Edge of water

Draw the cross-section of the river.

17. Choose the correct alternative for questions (i) through (x).

(i) A transit theodolite fitted with a stadia diaphragm and anallatic lens is known as a/an

(a) Tacheometer

(b) Subtense theodolite

(c) Astronomical theodolite

(ii) The stadia diaphragm is provided for measuring
(a) Elevation (b) Bearing (c) Horizontal distance

(iii) An anallatic lens is provided to make the additive constant equal to
(a) 100 (b) 0 (c) 90

(iv) The multiplying constant is denoted by
(a) $\frac{f}{i}$ (b) $\frac{i}{f}$ (c) $i \times f$

(v) The additive constant is denoted by
(a) $f - d$ (b) $f + d$ (c) $\frac{f}{d}$

(vi) When the line of sight is inclined and the staff is held vertically, the horizontal distance is given by
(a) $\frac{f}{i} \times S \cos^2 \theta + (f + d) \cos \theta$
(b) $\frac{f}{i} \times S \sin^2 \theta + (f + d) \sin \theta$
(c) $\frac{f}{i} \times S \cot^2 \theta + (f + d) \cot \theta$

(vii) Calculation of distance by application of the appropriate formula is very laborious. So, for easy computation a method is employed in field work which involves the use of a
(a) Logarithmic table
(b) Tacheometric table
(c) Computation table

(viii) The subtense bar is used to measure
(a) Vertical distance
(b) Horizontal distance
(c) Elevation

(ix) As the distance between the tacheometer and staff increases, the staff intercept by stadia hair
(a) Increases (b) Decreases (c) Remain constant

(x) In tangential tacheometry the staff is held
(a) Inclined
(b) Normal to the line of sight
(c) Vertically

ANSWERS

17.	(i) a	(ii) c	(iii) b	(iv) a	(v) b
	(vi) a	(vii) b	(viii) b	(ix) a	(x) c

12 Project Surveys

12.1 INTRODUCTION

Project work involves some technical terms, the meanings and purpose of which should be understood first. We shall now discuss a few useful terms.

1. The mass diagram Such a diagram is prepared to predict the proper distribution of excavated earth, position of borrowpits and the probable location of waste banks along the alignment of any project, which may involve roads, railways, irrigation, etc.

The diagram consists of a curve which is plotted with chainages as abscissae and the algebraic sum of volumes of cutting and filling as ordinates. The volumes of cutting and filling are counted from the starting point up to the point considered. The cuttings are taken as positive and fillings as negative.

In the case of a road in a hilly area, where there is a transverse slope, cutting and filling may occur at a particular chainage. In such a situation, the difference of cutting and filling is considered for construction of the mass diagram. The sign of the greater volume is taken into account while calculating the difference.

2. The balancing line This is the line which shows the equalising of cutting and filling in a particular length of the alignment. It is drawn parallel to the base line of the mass diagram.

3. Lead The horizontal distance through which the excavated earth from a borrowpit is carried to the place of embankment is known as lead (Fig. 12.1) As per the prevalent schedule of rates, a 30 m lead is considered normal. When lead exceeds this value, a higher rate is admissible.

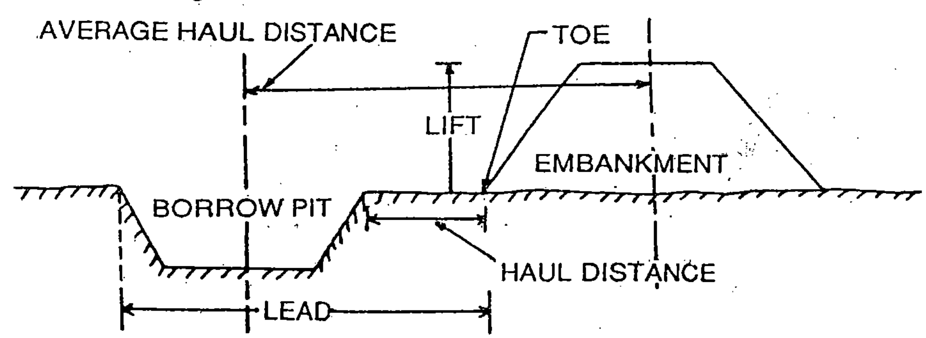


Fig. 12.1

4. Lift Lift is the vertical height through which the excavated earth from a borrowpit is to be lifted. As per the schedule of rates, a 1.5 m lift is considered normal, above which higher rates are admissible.

5. Haul distance The distance between the nearest face of cutting and the toe of the embankment is known as haul distance.

6. Average haul distance This equals the distance between the centre line (or centre of gravity) of cutting and the centre line (or centre of gravity) of embankment.

12:2 PREPARATION OF MASS DIAGRAM

The following procedure is to be adopted for preparing a mass diagram:

1. The volume of earth work (cutting or filling) is calculated according to the cross-section taken along the longitudinal section of the alignment.
2. Cutting is considered as positive and filling as negative.
3. The chainages along the longitudinal section are taken as the abscissa or base line which is plotted to a suitable horizontal scale.
4. The volume of cutting or filling is considered as the ordinate and plotted along the corresponding chainage to a suitable vertical scale.
5. The ordinates are joined by a smooth curve to get the mass diagram.
6. From the nature of the curve of the mass diagram, the distribution of excavated earth can be estimated.
7. A balancing line may be drawn to eliminate unnecessary cutting or filling.

Example A portion of the longitudinal section is given below, where the volume of earth work has already been calculated from the available field records. Draw the mass diagram.

Table for volume of earth work

Chainage (m)	Cutting (m^3) (+ve)	Filling (m^3) (–ve)
0–50		– 150
50–100	+ 120	
100–150	+ 90	
150–200		– 60
200–250	+ 100	
250–300		– 80
300–350		– 120
350–400	+ 130	

Solution Table for mass diagram

Section	Chainage (m)	Algebraic sum of cutting and filling	Net Volume (m^3)	Remarks
1	0	0	0	Starting point of work
2	50	0–150	– 150	Filling of – 150 m^3 between 0 to 50
3	100	– 150 + 120	– 30	Cutting of + 120 m^3 between 50 and 100
4	150	– 30 + 90	+ 60	Cutting of + 90 m^3 between 100 and 150
5	200	+ 60 – 60	0	Filling of – 60 m^3 between 150 and 200
6	250	0 + 100	+ 100	Cutting of + 100 m^3 between 200 and 250
7	300	+ 100 – 80	+ 20	Filling of – 80 m^3 between 250 and 300
8	350	+ 20 – 120	– 100	Filling of – 120 m^3 between 300 and 350
9	400	– 100 + 130	+ 30	Cutting of + 130 m^3 between 350 and 400

The mass diagram is shown in Fig. 12.2.

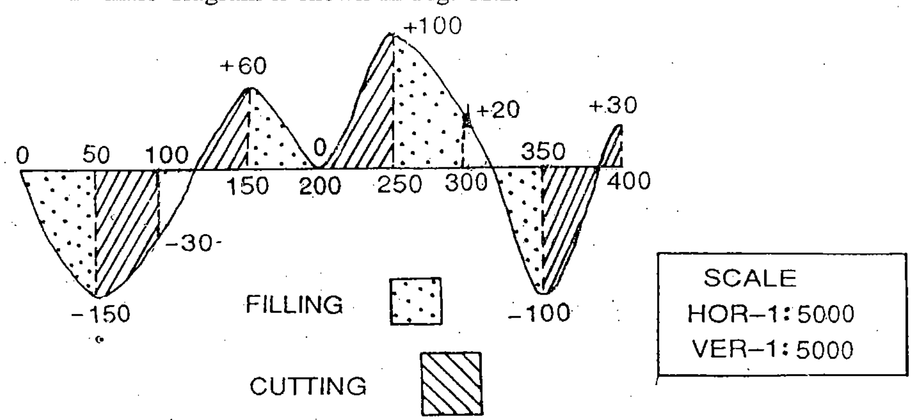


Fig. 12.2

12.3 RAILWAY PROJECT SURVEY

A railway project should be implemented in different stages, as described below:

A. Justification of Constructing New Line

When the question of constructing a new railway line between some places arises due to public demand or some other reason, an intensive study is carried out as regards viability of the project. The following points should be kept in mind while considering the justification of constructing a new line.

1. Total population of the villages, towns, industrial places, etc. coming under the project, should be recorded.
2. The standard of living and habits of the population, and the economic conditions of the locality, should be thoroughly studied as it will be a source of revenue for the department.
3. The amount of agricultural, natural and industrial resources should be recorded.
4. Information regarding religious places, religious fairs or festivals, business centres, etc. should be noted.
5. The amount of revenue that may accrue from passengers, agricultural goods, industrial goods, natural resources, etc. should be estimated.
6. A topographical map should be studied to determine the number of culverts, bridges, level crossings, tunnels, etc.
7. An agricultural map should also be studied so that too much valuable land is not affected.
8. A contour map should be studied in order to ensure economical alignment, avoiding unnecessary cutting or filling and maintaining a permissible gradient.
9. An Industrial map should be studied to find the shortest route to connect industrial areas.

B. Marking of Tentative Alignment

After the decision to set up a new railway line is taken, a tentative alignment or alignments are marked on the contour map and topographical map for the area concerned. While marking the tentative alignment, the following points should be kept in mind:

1. The route should be the shortest possible.
2. As far as possible, it should follow the ridge line to avoid unnecessary earth work in filling.
3. It should cross rivers perpendicularly.
4. It should not pass through religious places, such as temples, mosques, churches etc. or burial grounds, burning ghats, and so on.
5. The route should not pass through the centres of villages, towns, valuable structures, etc. where the compensation payable is likely to be more.
6. It should not be too zigzag, so that the number of curves is minimised.
7. It should not cross large depressions or valleys, high summits or ridge lines, to avoid huge earth fillings in areas of depression and tunnelling in regions including summits.

   If unavoidable, such areas are specially marked for finding suitable means to overcome these difficulties during the process of reconnaissance survey.
8. The tentative alignments of two routes should be marked for comparing costs of construction, feasibility, advantages, etc.

C. Reconnaissance Survey

For selecting a suitable alignment, a reconnaissance survey has to be conducted along the tentative alignments. The survey should be conducted for collecting the following data:

1. The magnetic bearings of the lines of the alignment are measured by prismatic compass, and the distances are measured by pacing (i.e stepping) and noted in a field book.
2. Approximate positions of different objects and the nature of the ground are noted in the field book, for a strip of land covering about 100 m on both sides of the alignment.
3. A soil survey should be conducted along the alignment.
4. Boring should be done along the alignment to know the nature of the water table.
5. The slope or undulation of the ground should be determined by Abney level or hand level.
6. The number of crossing points, like roads, rivers, canals, etc. should be noted. Detailed information regarding these points should be recorded, so that it is possible to design suitable structures to cross them.
7. The number of curves and difficulties in curve setting, if any, should be clearly noted.
8. The number of culverts or bridges should be noted along with detailed information pertaining to span, HFL, discharge, etc.
9. Rainfall records of the area and the discharge records (for the past 10 years) of big rivers should be collected.
10. Large depressions and high summits should be avoided as far as possible to minimise the cost of earth work or tunnelling. Where they are unavoidable, detailed information regarding these obstacles should be recorded.
11. Availability of construction material and the possible route for their collection should be noted.
12. Availability of labour, suitable places for labour colonies, etc. should be recorded.
13. A project report should be prepared accompanied with an index map of alignment, approximate longitudinal section, number of curves, culverts, bridges, etc. to select a suitable alignment for preliminary survey.
14. A preliminary record of properties for which compensation is to be paid along with such details as the name of the owner, quantity of land, present valuation, etc. should be prepared.

D. Selection of Good Alignment

After reconnaissance survey, a good alignment or alignments are selected for preliminary detailed survey. While selecting such alignment, the following points should be kept in mind:

1. The route should be the shortest possible, and economical.
2. It should pass through important places, for it to be possible to earn considerable revenue from passengers, export of goods, etc.
3. The route should connect areas rich in minerals, agricultural resources, forests, etc.

4. The number of culverts and bridges should be minimum and the width of the river at the crossing point should be the shortest.
5. Deep cutting, excessive banking and tunnelling should be avoided.
6. Vertical curves should be avoided.
7. The selected station yards should be easily accessible to surrounding localities.
8. Availability of construction material, labour, etc. is also an important factor.
9. A comparative analysis should be made as regards cost of construction and revenue expected.

E. Detailed Preliminary Survey

After selection of suitable alignment or alignments, a detailed preliminary survey is conducted. Such survey should be extremely accurate as selection of the final alignment depends on it. Preliminary survey involves the following steps:

1. The starting point of the project is marked by a constructing pillar.
2. Fly levelling is done to connect the nearby GTS benchmark with the starting point of the project.
3. A prismatic compass survey is conducted along the alignment to prepare a route survey map covering about 100 m on both sides of the alignment. Sometimes a plane table survey is conducted in order to prepare the route survey map.
4. Longitudinal levelling is done along the alignment at regular intervals (say of 20 or 40 m). The magnetic bearings of the lines of traverse (open traverse along the alignment) should be noted in a level book.
5. Cross-sectional levelling is done at regular intervals (say of 100 m).
6. Permanent benchmarks should be established at regular intervals (of say 2 km) along the alignment.
7. At the points of river crossing the cross-sections of the river are taken at different points at regular intervals (say 100 m) upstream and downstream for a distance of about 1 km from the bridge site. Additional information, such as width of river, velocity of water, discharge, HFL, scour depth, etc. should be noted.
8. A contour survey is done on the marked station yards and a plane table survey map is also prepared for these yards.
9. A soil survey is undertaken along the alignment to know the bearing capacity of the soil and the nature of the water table.
10. River beds are bored in order to find the depth of foundation for the bridge pier.
11. *Preparation of Drawing* The drawing prepared should include the following.
    (a) Route survey may of the strip of land showing details of objects and nature of ground along the alignment.
    (b) Longitudinal section of the alignment showing the formation level.
    (c) Cross-sections showing the formation width and side slopes, of cutting or banking.
    (d) Contour map of the strip of land along the alignment.
    (e) The total land width required for the project is marked on the route survey map.
    (f) Design of curves with setting out table.
12. Preparation of Estimate:
    (a) Earth work required in cutting and banking is estimated.
    (b) A mass diagram is prepared for balancing the cutting and banking operations.
    (c) An approximate estimate is done for culverts, bridges, level crossings, etc. (A final estimate is prepared after the design.)
    (d) A compensation list is prepared showing full details.
    (e) A detailed costing of the project is then prepared for inclusion in the project report.

F. Final Location Survey

Of the alignments considered in the preliminary survey, the most economical is selected. Before the approval of the project, the final location survey is completed in all respects. This survey helps the engineers in charge of construction projects. The final location survey involves the following stages:

1. The centre line of the alignment is marked with masonry pillars at regular intervals. Generally, stout pegs are provided every 30 m and masonry pillars every 1 km.
2. The total land width required is properly demarcated by pillars.
3. The tangent points and intersection points of the curve are marked with pillars having proper designation.
4. The station yards are marked at the required points.
5. The points of level crossings are marked showing the total land width.
6. The points of culverts are marked properly.
7. Bridge sites are marked with pillars on both banks of the river.
8. A final record of properties for which compensation is to be paid, is prepared.

G. Project Report

After completion of all survey work, a project report should be prepared and submitted for approval. The report should include the following matter:

1. Introduction to the project.
2. Necessity and background of the project.
3. Justification of selection of final alignment and the procedure adopted for land acquisition.
4. Details of proposed alignment such as gauge, length covered, area covered, number of culverts, bridges, level crossings, tunnels, etc.
5. Detailed estimate of the project covering all items—earth work, construction of culverts, bridges, tunnels, station yards, compensation, etc.
6. Details of specification during construction.
7. Expected revenue from the project.
8. Conclusion and recommendation for the project.

9. Maps to be submitted with the project report:
   (a) General map of the country through which the proposed line will pass (scale—1 cm = 20 km),
   (b) Route survey map (to suitable scale),
   (c) Longitudinal section (to suitable scale),
   (d) Cross-section (to suitable scale),
   (e) General map and contour map of station yards (to suitable scale),
   (f) Detailed drawings of culverts, bridges, tunnels, level crossings, etc. (to suitable scale), and
   (g) Complete drawings of station buildings, station yards, etc. (to suitable scale).

12.4 ROAD PROJECTS

A. Necessity

When the question of constructing a new road arises due to public demand or some strategic reason, a primary investigation is carried out to examine whether this road is necessary. The following points are to be kept in mind at the time of such investigation:

1. Total population benefited by the project.
2. Number of villages, towns, industrial places, etc. to be connected.
3. Agricultural products, industrial products, minerals, etc. are likely to be conveyed through the proposed road and thus help the development of trade in the country.
4. Prospect of tourism, if any.
5. Strategic importance for the defence of the country.
6. Any other information related to the project should be noted.

B. Marking the Tentative Alignment

After primary investigation regarding the justification of constructing a new road, the tentative alignment or alignments are marked on the general map and contour map of the area through which it is expected to pass. While marking the tentative alignment, the following points should be considered:

1. The proposed road should connect a sufficient number of villages, towns, industrial places, places of religious importance, etc.
2. The alignment should be taken in such a way that unnecessary cutting and banking can be avoided.
3. If the alignment crosses a river, it should do so perpendicularly through the shortest width of the river.
4. The alignment should not pass through religious places like temples, churches, mosques, etc. or burial grounds, burning ghats and so on.
5. The alignment should not be taken completely through valuable agricultural land.
6. The alignment should not pass through the heart of villages, towns, etc. where the compensation payable is likely to be more.
7. Any other obstacles or problems should be considered during reconnaissance survey.

C. Reconnaissance Survey (Reconnoitre)

Before starting the actual survey work, a reconnaissance survey is conducted along the tentative alignments to select the most suitable alignment. The following points should be noted during reconnaissance:

1. The magnetic bearings of the lines of alignment are measured by prismatic compass and noted in a field book.
2. The distances along the alignment are measured approximately by pacing (one pace or walking step is taken as 80 cm or 2.5 ft).
3. The objects and nature of the ground on both sides of alignment, up to 50 m, are noted in the field book.
4. Obstacles like religious places or valuable structures, if any, should be suitably crossed over.
5. If the tentative alignment crosses a river obliquely or passes through a wide cross-section of it, then the alignment is diverted suitably to cross the river perpendicularly and through its shortest width.
6. All other important points like railway crossings, canal crossings, etc. should be noted.
7. The highest flood level ever attained and the discharge records for the last 10 years should be collected from the appropriate authorities to design the culverts and bridges.
8. Preliminary records should be prepared of properties eligible for compensation.

D. Preliminary Location Survey

After reconnaissance survey, a suitable alignment or alignments are selected for preliminary location survey for detailed investigation to obtain the most economical alignment. The preliminary survey is done in the following way:

1. The starting point of the project is marked by a pillar.
2. Fly levelling is done to connect the nearby GTS or permanent BM with the starting point of the project.
3. A prismatic compass survey is conducted to prepare a route survey map covering about 50 m on both sides of the alignment.
   Sometimes a plane table survey is done in order to obtain the route survey map.
4. Longitudinal levelling is done along the alignment at regular intervals (of say, 20 or 40 m).
   The magnetic bearing of each line should be noted in the level book.
5. Cross-sections are taken at regular intervals (of, say, 100 m).
6. Permanent benchmarks should be established at suitable places along the alignment for future reference.

7. The cross-sections of rivers, *nullahs*, etc. should be taken accurately.
8. In the case of big rivers, additional data should be collected for designing the bridges. The following points should be kept in mind:
   - (a) Cross-sections at 100 m intervals are taken about 500 m upstream and 500 m downstream of the bridge site.
   - (b) HFL ever attained should be noted.
   - (c) Boring should be done on the river bed to find the depth of foundation of the piers.
   - (d) The nature of erosion and scour should be noted.
9. Preparation of drawing: The drawing prepared should include the following:
   - (a) Route survey map (to suitable scale)
   - (b) Longitudinal section with formation level (to suitable scale)
   - (c) Cross-sections with formation width and side slopes
   - (d) Contour map of the strip of land along alignment
   - (e) Design of curves with setting out table
   - (f) A mass diagram for the earth work
10. Office work:
    - (a) Total land width required is marked on the route survey map
    - (b) Estimate of earth work in cutting and banking
    - (c) Design and cost estimate of culverts and bridges
    - (d) Cost estimate of fly over, if any
    - (e) Estimate for compensation required
    - (f) Estimate of road surface construction
    - (g) Total cost of project for the tentative alignment

E. Final Location Survey

The most economical alignment is selected by analysing the merits, demerits, cost of construction, etc. for the proposed alignments after preliminary location survey. Before the approval of the project is obtained from the higher authorities, the final location survey is completed in all respects. The following steps are taken for final location survey:

1. The centre line is marked by stout pegs or pillars at intervals of 30 m.
2. The total land width required is marked by pillars at regular intervals (of, say, 30 m).
3. The tangent points and intersection points of the curves are properly marked by pillars.
4. A final record is prepared of properties eligible for compensation.

F. Project Report

After completion of all investigation work, survey work design of different structures, and total estimate for the project, a report should be prepared and submitted to the higher authorities for approval. The report prepared should include information related to the following:

1. Introduction to the project
2. Necessity and background of the project
3. Justification for selection of the final alignment and the procedure adopted for land acquisition
4. Detailed estimate covering all items—earth work, road surface, culverts, bridges, compensation, etc.
5. Detailed specification for the constructional works
6. Overall benefit of the project
7. Conclusion and recommendation
8. Maps to be submitted along with project report:
   - (a) General map of the country through which the proposed road will pass
   - (b) Route survey map (to suitable scale)
   - (c) Longitudinal section (to suitable scale)
   - (d) Cross-section (to suitable scale)
   - (e) Detailed drawing of culverts, bridges, flyovers, etc

12.5 PROJECT SURVEY ON FLOW IRRIGATION

A. Necessity

Before an irrigation project is taken up, it should be examined whether the area concerned in fact needs to be irrigated. The investigation should relate to the following points:

1. Amount of yearly rainfall
2. Nature of distribution of rainfall during the year
3. Types of crops grown in the area
4. Water requirement of the crops
5. Whether the rainfall can meet the requirement or not
6. Future prospects if irrigation is practised

B. Availability of Irrigation Water

When it is found necessary to implement an irrigation system, the availability of irrigation water should be investigated. The following points should be kept in view:

1. Whether any perennial river is available
2. If an inundation river is available, the yearly discharge and nature of the river should be studied
3. Whether there is a suitable site for a weir, dam or barrage
4. Whether the river can meet the total requirement of water

C. Study of Topography

The topography of the area concerned should be studied and the country slope should be examined carefully for the marking of tentative alignment. The nature of cultivable area should be noted.

D. Selection of Site for Dam, Barrage or Weir

When the source of water is selected, the possible site of the dam should be investigated keeping in mind the following points:

1. A good foundation should be available for the dam. The required investigations are carried out by boring, pile testing, etc.
2. A suitable basin should be available for the storage reservoir.
3. The capacity of the reservoir should fulfil the requirement.
4. The reservoir should not submerge considerable area and valuable regions.
5. Construction material and labour should be readily available.
6. The allowable bed slope of the canal should be maintained as far as possible.

E. River Gauging

The discharge observation site (river gauging) should be established at the proposed dam site to collect the following data:

1. The daily and yearly discharge, highest flood level, and lowest water level of the river should be recorded.
2. Silt analysis should be done in order to gather information about the possible silting of the river bed and the manurial value of the fine silt.
3. River gauging should also be done for all the rivers through which the proposed canal may cross. This is for collecting data for cross-drainage works.

F. Marking of Culturable Command Area

After various investigations, when it is decided to take up the project, the culturable command area is marked on the topographical map.

G. Marking of Tentative Alignment

The tentative alignment or alignments are marked on the topographical map and contour map of the area concerned. While marking such alignment the following points should be remembered:

1. The canal alignment should be taken in such a way that unnecessary cutting and banking can be avoided.
2. The alignment should be such that branch canals can be suitably cut to cover the whole area.
3. The canal alignment should cross rivers, roads, railway lines, etc. perpendicularly.
4. The canal should not be cut through valuable agricultural land.
5. The canal should not pass through religious places.

H. Reconnaissance Survey

To select the suitable alignment or alignments, a reconnaissance survey is done along the tentative alignments to record necessary data related to obstacles, road

crossings, railway crossings, river crossings, etc. to facilitate preliminary location survey. Reconnaissance involves the following procedure:

1. The magnetic bearings of the lines of the traverse are noted.
2. The approximate distances are measured by pacing.
3. The objects and nature of the ground up to a distance of 100 m on both sides of the alignment are noted in the field book.
4. The nature of the country slope is recorded.
5. The tentative alignment may be diverted to avoid religious places, valuable structures etc.
6. The canal alignment is made to cross rivers perpendicularly and across their shortest width.
7. The road crossings and railway crossings should also be made perpendicular.
8. The types of suitable cross-drainage works should be noted.
9. The highest flood levels ever attained by the rivers should be noted by examining relevant records or collecting information from villagers.
10. A preliminary record should be prepared of properties eligible for compensation.

I. Preliminary Location Survey

After reconnaissance survey, a suitable alignment or alignments are selected for preliminary survey. The preliminary survey has to be extremely accurate as the estimate of cost, final location, etc. depend entirely on it. Preliminary survey involves the following steps:

1. The centre line of the dam, barrage or weir is marked with pillars on both banks of the river.
2. The centre line of the head works for the left bank canal or right bank canal is marked with pillars.
3. Fly levelling is done to connect the GTS benchmark to any permanent point near the dam site.
4. The cross-section of the dam site is taken precisely. Then a number of cross-sections are taken both upstream and downstream of the dam site for about 1 km.
5. The river bed is bored along the centre line of the dam to determine the depth of foundation.
6. The starting point of the canal is connected with the permanent benchmark established previously near the dam site.
7. A prismatic compass survey or plane table survey is conducted along the alignment to prepare a route survey map for the area about 100 m on both sides of the alignment.
8. Longitudinal levelling is done by taking levels at regular intervals (of, say, 20 m) and the magnetic bearings of all the lines are noted in a level book.
9. The cross-sections are taken at regular intervals (of, say, 100 m).
10. During longitudinal levelling temporary benchmarks are established after the end of the day's work and check levelling is done to check the work

of that particular day. Permanent benchmarks are established at suitable points along the alignment for future reference.

11. At the places of road crossings, railway crossings, etc. additional data should be collected for designing cross-drainage work.
12. At the point of river crossing, cross-sections of the river are taken upstream and downstream for about 500 m for designing the cross-drainage work.
13. A soil survey is conducted along the alignment.
14. Well observations are taken along the alignment about 200 m on both sides of the alignment for information about the water table.
15. The longitudinal and cross-sections of the branch canal are also taken.
16. Preparation of drawings: The drawings prepared should include the following:
    (a) Route survey map (to suitable scale)
    (b) Longitudinal section with formation level (to suitable scale)
    (c) Cross-sections with formation level (to suitable scale)
    (d) Contour map along alignment
    (e) Design of curves with setting out table
    (f) Mass diagram for earth work
17. Office work:
    (a) An estimate is prepared of the earth work required for cutting and banking for the main and branch canals. The total land width required is marked on the route survey map.
    (b) A design and estimate is prepared for the dam, head works, cross-drainage works, etc.
    (c) An estimate of payable compensation is made.
    (d) The total cost of the project is calculated.

J. Final Location Survey

The most economical alignment is selected by comparing the costs of construction and overall benefits of the proposed alignments after the investigations in the preliminary survey. Before the project report is prepared and submitted to the higher authorities for approval, the final location survey is completed in all respects. Such survey involves the following steps:

1. The centre line of the canal is marked with stout pegs or pillars at regular intervals (of, say, 30 m).
2. The total land width required is marked with pillars.
3. The centre line of the branch canal and the total land width required are marked with pillars.
4. A final record is prepared of properties eligible for compensation.

K. Project Report

After completion of all investigation and design work, and preparation of final the estimate, a project report is prepared and submitted to the higher authorities for approval. The report should include information related to the following:

1. Introduction
2. Necessity and economical justification of the project
3. Justification for selection of final alignment and procedure adopted for land acquisition
4. Detailed estimate for all such items as earth work, dam, head work, cross-drainage work, compensation, etc.
5. Detailed specification for constructional work
6. Overall benefit of the project
7. Conclusion and recommendation
8. Maps to be submitted:
    (a) General map of the country through which the proposed canal will pass (scale—1 cm = 20 km or any suitable scale)
    (b) Route survey map (to suitable scale)
    (c) Longitudinal section (to suitable scale)
    (d) Cross-sections (to suitable scale)
    (e) Contour map of alignment (to suitable scale)
    (f) Detailed drawing of dam, head work, cross-drainage work, etc.

12.6 PROJECT SURVEY ON WATER SUPPLY SCHEME

A. Reconnaissance

When a water supply scheme is to be prepared for a newly developed town or city or an already existing scheme has to be expanded, a primary investigation has to be conducted on the concerned area and a primary report prepared regarding implementation of the scheme. During reconnaissance the following points should be noted:

1. Total area to be covered
2. Existing population, habits of people, types of industries, etc.
3. Existing sources of water for drinking and other purposes
4. An index sketch is to be prepared showing the population densities at different zones
5. Trend of development of the town or city
6. Intensity of public demand for the water supply scheme

B. Demand of Water

A water supply scheme is never prepared only for the present population. It should be designed to serve the probable population after at least three decades. So while taking up a water supply scheme, the demand of water for the next three decades should be ascertained first. The total water demand is estimated considering the following points:

1. The present population is determined and then the population for the next three decades is estimated by the usual methods.
2. The daily rate of water demand is worked out depending on the habits of the people, types of industries, sewerage system, fire demand and other factors.

3. Then the total demand for the peak hours is estimated depending on the method of water supply.

C. Source of Water

After computation of the demand of water for the scheme, an investigation is carried out for finding a suitable source of water which is adequate in respect of quantity and quality. The investigation is carried out based on the following considerations.

1. Surface source The surface source may be in the form of a perennial river, inundation river, large lake, etc.

In a perennial river, the quantity of water may be adequate, but the quality should be examined in order to establish that excessive treatment is not required.

In an inundation river, the quantity is not adequate. In this case, an artificial reservoir has to be prepared by constructing a weir across the river. Sometimes a dam or barrage may be constructed for this purpose.

In a large lake, the quality may be reliable, but the quantity should be examined for adequacy by studying the water level throughout the year.

Again, a suitable intake point should be located so that a system of supply based on gravity may be implemented.

If the country slope does not permit implementation of a gravity system, supply has to be through an elevated reservoir. In such a case, a pump house has to be constructed.

2. Underground source If an underground source is located and found to be adequate in all respects, deep tube wells should be sunk at suitable points in different zones. In such a case, water is supplied through a direct pumping system.

D. Preparation of Topographical Map

For implementation of a water supply scheme, a topographical map of the town or city has to be prepared. The topographical map should indicate the location of roads, railways, houses, ditches, high grounds, etc. The nature of the ground surface is represented in relief by colour shading, contour lines, hachures, etc. (Methods of preparing relief maps are discussed in Sec. 12.11.)

E. Layout Map of the Scheme

On the prepared topographical map, the layout of the scheme is marked by using a different colour convention or any suitable convention so that the work can be conducted in different phases. The layout should include the following information:

1. The zone and type of intake work
2. The zone and units of the treatment plant
3. The method of conveyance from the intake point to the treatment plant
4. The zone and type of storage reservoir
5. The network of main pipe lines
6. The network of the distribution system
7. Specific points such as positions of check valves, fire hydrants, inspection chamber, junction points, etc.
8. The location of pumphouses for deep tube wells

F. Maps and Drawings to be Prepared

The following maps and drawings have to be prepared for the scheme.

1. A topographical map of the town or city (to suitable scale)
2. A layout map of the scheme (to suitable scale)
3. Detailed drawings of intake works, pump houses, overhead reservoir, etc.

G. Office Work

The office work required includes preparation of estimates for the following.

1. Intake work, purification plant, overhead reservoir, pump houses, pipelines, etc.
2. Compensation payable
3. Other allied expenditure
4. Total cost of the scheme

H. Project Report

When all the investigation work design and drawing, estimation etc. have been completed, a report should be prepared and submitted to be higher authorities for approval. The report should contain information related to the following:

1. Introduction
2. Necessity and background
3. Justification of taking up the present scheme and procedure adopted for land acquisition
4. Detailed estimate of the scheme
5. Detailed specification for constructional work
6. Conclusion and recommendation
7. Maps to be submitted:
   (a) Topographical map
   (b) Layout map of the scheme
   (c) Detailed drawing of intake work, treatment plant, pump houses, overhead reservoir, etc.

12.7 PROJECT SURVEY ON SANITARY SCHEME

A. Reconnaissance

When a sanitary scheme is to be prepared for a newly developed area of a town or city or some modifications have to be made to the existing system, a reconnaissance survey should be conducted in order to evaluate the feasibility of the scheme. During reconnaissance the following points should be noted:

1. The total area to be covered under the scheme has to be worked out.
2. The present population, habits of population, nature of colonies, public places, etc., have to be considered.
3. An index map has to be prepared showing the different zones, public places, nature of colonies, trend of development of the town, etc., and
4. A population forecast has to be made for the next three decades.

B. Topographical Map

The topographical map of the town or city is prepared, showing the different colonies, housing estates, high value premises, public places, roads, ditches high grounds etc. using different colour shades or symbols. (The method of topographic survey is described in Sec. 12.11.)

C. Division into Different Zones

For successful implementation of the scheme, the area of the town under the scheme is divided into different zones. This is done because the method of garbage collection and sewerage system may vary from zone to zone. Using some colour convention, the different zones are marked on the topographical map.

D. Skeleton Map for Garbage Collection

To ensure easy disposal of garbage, the positions of garbage collection centres, dustbins, etc. should be marked on the topographical map, so that the vehicles employed for collection and disposal may move easily without causing any inconvenience to the public.

The location of the dumping place should be marked on the skeleton map at a suitable area for away from the locality. A specific route should be indicated for the collection and transport of garbage.

E. Skeleton Map of Underground Sewer Line

Nowadays, a water carriage system is considered essential for disposal of sewerage, for reasons of hygiene and convenience. Therefore a sewerage system has to be implemented for which a skeleton map of the underground sewer line has to be prepared. The following points should be borne in mind while preparing the skeleton map:

1. In the main sewer line, specific points like manholes, lampholes, inspection chambers, etc. should be marked.
2. In the branch sewer line, specific points of importance should be marked.
3. At the point of change of grade, the location of the pump house should be marked, if one is required.
4. The position of the treatment plant should be shown.
5. The points of disposal of effluents and the area of sludge disposal should also be indicated.

F. Preparation of Maps and Drawings

The following maps should be prepared for the scheme:

1. Topographical map (to suitable scale, showing different zones)
2. Skeleton map of garbage collection
3. Skeleton map of underground sewer line
4. Design and drawing of treatment plant
5. Detailed drawing of pump house

G. Office Work

The office work required involves preparation of estimates for the following:

1. The underground sewer line
2. The treatment plant
3. The purchase of vehicles for garbage collection
4. Compensation payable
5. The total cost of the scheme

H. Project Report

After completion of all investigation work, design work, estimate, etc., a report has to be prepared and submitted to the higher authorities for approval. The report should include information on the following:

1. Introduction
2. Necessity
3. Justification of taking up the present scheme, and procedure adopted for land acquisition
4. Detailed estimate of the scheme
5. Detailed specification for the construction
6. Conclusion and recommendation
7. Drawings to be submitted:
   (a) Topographical map
   (b) Skeleton map for garbage collection
   (c) Skeleton map of underground sewer line
   (d) Detailed drawings of treatments plant, pump house, etc.

12.8 PROJECT SURVEY ON DOCKS, HARBOURS AND PORTS

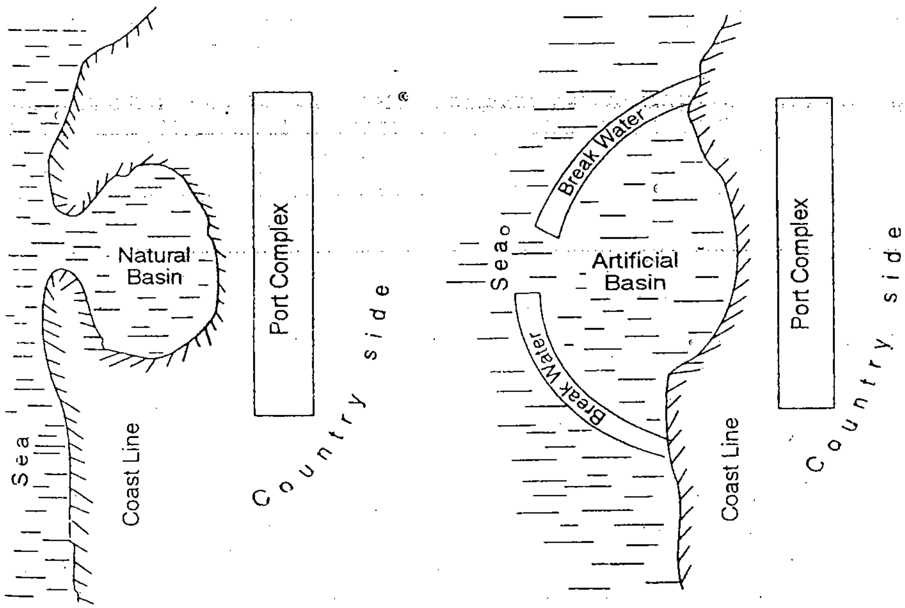
A. Introduction

A port complex consisting of docks and harbours is essential for coastal cities for purposes of inland and international navigation. Before taking up a scheme for setting up a port complex, necessary investigations have to be conducted in order to establish a suitable location for it. Such investigation involves various phases, which are dealt with one by one.

B. Reconnaissance Survey

When it is required to prepare a scheme for a new port complex, or when an already existing port complex has to be modified, a reconnaissance survey is conducted primarily for finding the approximate location of the site. The reconnaissance survey should be based on the following points:

1. The shore line should not be straight at the site of the port complex, but in the form of loop so that the port complex can be protected from tides and waves. Ideal conditions are shown in Fig. 12.3(a) and (b).



(a) Natural Basin (b) Artificial Basin

Fig. 12.3 (a) and (b)

2. The depth of water near the shore should be more than 10 m.
3. The tidal effect, wave and wind effect on the shore should be comparatively low.
4. Considerable open area should be available near the shore for construction of the terminal building and other allied structures.

C. Coastal Survey

For natural protection of the port complex, the shore line should be of a particular shape. So, to find a suitable site, an open traverse is conducted with prismatic compass, theodolite or plane table along the coast line. The exact shape of the coast is determined by taking offsets from the traverse legs. A map is prepared for this survey to an appropriate scale. Before the survey work, the shore line during low tide and high tide is demarcated. From the coastal survey map, a suitable site is selected.

D. Topographic Survey

A Topographic map has to be prepared for a strip of land of width about 1 km along the coastal line. This map should show the existing structures, roads, various objects and the nature of the ground. To indicate the nature of the ground, the topographical map is drawn in relief. Topographic survey is discussed in more detail in Sec. 12.11.)

E. Hydrographic Survey

Such survey is conducted to measure the depth of water near the shore line, identify obstacles near the shore, and determine the variation of the shore line during low and high tides. Hydrographic survey is conducted according to the following steps:

1. A base line is taken along the shore line and sections are marked perpendicular to it at regular intervals (of, say, 30 m). Then the depth of water at regular intervals (of, say, 10 m) is found out by fathometer or echo sounder (described in Chap. 13). A contour map is prepared to know the nature of the sea bed.
2. The water levels attained to low and high tides should be noted and so should the scouring effect of the waves. A tide gauge is established at the shore to note the variation.
3. The velocity of water current is measured by current meter or subsurface float.

F. Soil Survey

A stable foundation is essential for marine structures. Therefore, an intensive soil survey (i.e., soil investigation) should be conducted along the coastal line. In such survey, boring is done along the shore line and on the sea bed at regular intervals and at specific points. The results of the soil investigation are studied thoroughly in order to ensure a stable foundation for marine structures.

G. Selection of Site

Considering all the essential factors for an ideal port complex, a suitable site is selected. The master plan of the complex is prepared to suit in the selected site. Selection of the site depends on:

1. The shape of the coast line
2. The depth of water near the coast
3. The wave and tidal effect on the coast
4. The presence of a stable foundation for marine structures

H. Structural Design

The master plan of the complex is oriented and the various structures for the complex designed according to site conditions. Detailed drawings of all the structures are prepared.

I. Estimate

Estimates have to be prepared for the following.

1. Construction of marine structures and all other allied structures,
2. Compensation payable, and
3. Total cost of construction.

J. Project Report

After intensive investigation, when all the work is completed, a project report is prepared and submitted to the higher authorities for approval. The project report should cover the following points:

1. Introduction
2. Necessity
3. Justification of selecting the site and procedure adopted for land acquisition
4. Detailed estimate for the project
5. Detailed specification for construction
6. Conclusion and recommendation
7. Supporting drawings:
   (a) Coastal survey map (to suitable scale)
   (b) Topographical map (to suitable scale)
   (c) Master plan of the complex (to suitable scale)
   (d) Details of structural design and drawing

12.9 PROJECT SURVEY ON AIRPORT

A. Necessity

Air transport is the fastest of all transport systems. Nowadays, it plays an important role in the development of a country. To facilitate rapid movement of goods and passengers, all important cities of a nation are connected by air. The construction of airport complexes is obviously essential for implementation of an air transport system. The following points should be studied while examining the need for an airport in a particular city or region.

1. Total population
2. Standard of living of population
3. Industrial, commercial and natural resources
4. International value of these resources
5. International importance of the place
6. Inland importance of the place
7. Strategic importance

B. Availability of Area

When it is felt necessary to establish an airport in a particular city or region, the availability of required area is the main consideration. An airport complex requires a large area which has to accommodate a terminal building, runway, taxi way, apron, hangar, etc. So a primary investigation has to be conducted around the city to locate a suitable area.

C. Reconnaissance Survey

When a suitable area has apparently been located, a reconnaissance survey is conducted there to collect necessary data for further investigation which would involve preliminary survey, soil survey and so on. The reconnaissance survey should be based on the following points:

1. Extent of area
2. Mode of communication with the concerned city
3. Types of obstructions, difficulties, etc.
4. Configuration of the area
5. Preparing a record of properties eligible for compensation and estimating the type of opposition that may occur

D. Preliminary Survey

A preliminary survey is conducted in order to prepare a general map of the area. This map should show details of objects such as buildings, roads, railways, rivers, ponds, etc. The nature of the ground surface is illustrated in a contour map prepared separately. The preliminary survey should be conducted in the following way:

1. A theodolite traverse is done covering the whole area, and the interior details are located by plane table. (Theodolite is discussed in detail in Chapter 9.)
2. A contour map is prepared by an indirect method, by taking spot levels with a levelling instrument. (The procedure is described in Chapter 6.)
3. If the area is very irregular, unsuitable for chaining, and extremely tedious to work on, tacheometric surveying is done to prepare the contour map. (The procedure is given in Chapter 11.)

E. Soil Survey

The design of the airport complex depends on the prevalent subsoil conditions. Therefore, a soil survey has to be carried out in order to know the nature of the subsoil in the area. Samples of soil at various depths are collected and tested in the soil testing laboratory. If suitable soil is not available, then it may be stabilized by appropriate methods. Generally the soil samples are collected by Auger boring or wash boring, or from test pits.

F. Meteorological Survey

This survey is conducted to know the details of wind direction throughout the year, amount of rainfall, nature of morning and evening fog or frost in winter, nature of snowfall (in hilly areas) and so on. The direction of wind is a vital factor to be considered while deciding the direction of runways.

G. Selection of Site

A suitable site is selected after examining the factors discussed above.
The master plan of the airport complex is prepared according to site conditions. Then this plan is oriented on the prepared map of the selected site.

The various structures, such as the terminal building, hangar, etc., are designed. The constructional details of the runway, taxiway, apron, etc. are worked out. Then the necessary drawings for these structures are prepared.

H. Preparation of Estimate

An estimate has to be prepared of the expenditure on the following:

1. Payment of compensation
2. Terminal building, hangar, runway, etc.
3. Other allied expenditure
4. Total cost of construction

I. Project Report

After all the necessary investigations have been carried out, a project report is prepared and submitted for departmental approval. The report should cover

1. Introduction
2. Necessity
3. Justification of selection of the site
4. Details of procedure adopted for land acquisition and settlement of compensation claims
5. Detailed estimate of the project
6. Detailed specification of construction
7. Conclusion and recommendation
8. Maps to be submitted:
   (a) General map of the area (to suitable scale)
   (b) Contour map of the area (to suitable scale)
   (c) Layout of master plan (to suitable scale)
   (d) Details of structural design and drawing (to suitable scale)

12.10 TUNNELLING

A. Necessity

A tunnel is most essential when the alignment of a railway track passes through

a mountainous region. A railway line should always be laid along a specified gradient. So, in a hilly area where a specified grade cannot be maintained due to the nature of the hill surface, a tunnel is the only solution. In addition, a tunnel has to be constructed in the following situations

1. When the cost of an open cut (i.e. excavation) in a mountainous area is very uneconomical,
2. When the route around a mountain is too long,
3. When bridge construction across a river becomes uneconomical, and
4. When in large and congested cities an underground railway is considered to be the only possible means of rapid transportation.

B. Marking of Alignment

When tunnelling is found to be the best solution to laying a railway line through a mountainous region, the following points should be kept in mind while marking the alignment:

1. The alignment should pass through the shortest width of the ridge.
2. The type of soil or rock should be suitable for tunnelling.
3. The grade should be uniform within the tunnel and on the approach of both ends.

C. Location of Centre Line on Existing Hill Surface

The centre line of the tunnel should be marked with extreme accuracy. This is because a slight error in it may lead the tunnelling operation in a wrong direction, in which case all the work would be in vain. For short tunnels, the centre line is marked by vernier theodolite, and for long tunnels by micrometer theodolite.
The procedure of marking the centre line is as follows: (See Fig. 12.4.)

1. The theodolite is set up at T_1, a point previously fixed on the alignment from the preceding station T. The theodolite is then levelled with respect to the plate bubble and the altitude bubble. Vernier A is set at zero and a backsight is taken on the backward station T. Both the clamps are lightened.
2. The telescope is transited, and by raising it the ranging rods at P_1, P_2 and P_3 are bisected and the points are marked on the ground by stout pegs or stone pillars.
3. The theodolite is shifted and centred over P_3. After proper temporary adjustment, a backsight reading is taken on the ranging rod at T_1. Both the clamps are fixed. The telescope is transited and the ranging rods at P_4 and P_5 are bisected. The points are then properly marked on the ground.
4. Again the theodolite is shifted and centred over P_5, and the points P_6, P_7, ... marked according to previous procedure.
5. Thus the points P_1, P_2, P_3, ... are on the alignment (centre line of the tunnel).
6. Levelling is done along the centre line and the reduced levels of the points P_1, P_2, P_3, ... are calculated.

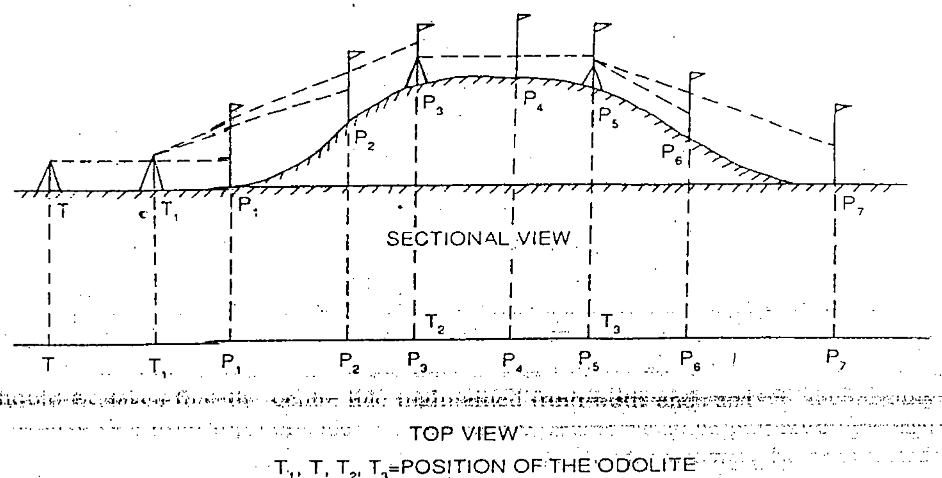


Fig. 12.4

At this stage, permanent benchmarks are establishment along the centre on top of the hill for future reference.

D. Transfer of Centre Line Inside Tunnel (through shaft)

In the case of small tunnels, the excavation is started from one or both ends, and the centre line of the tunnel is maintained by a theodolite set up on one or both ends of the tunnel.

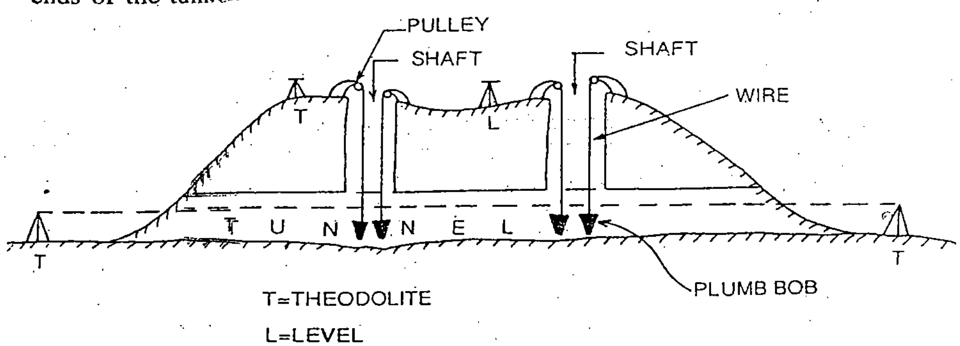


Fig. 12.5

But in the case of long tunnels the excavation is started from both ends and also from some intermediate points through a shaft, which may be dug from the top or from one side.

The centre line of the tunnel is required to be transferred underground to locate the axis of the tunnel. During excavation, the centre line maintained from the ends and the one transferred through the shaft should coincide.

The following procedure is adopted for transferring the centre line from top:

1. A theodolite is set up on top of the hill at a suitable position to maintain the centre line of the shaft.
2. With the help of a levelling instrument, the RLs of both ends of the shaft are determined. Since the bottom level of the tunnel is already determined, the exact depth of the shaft is calculated with reference to the top RL.
3. Then the excavation of the shaft is started, and verticality is maintained by a plumb-bob suspended from a wire.
   Excavation is continued until the calculated bottom level is reached. The depth of the shaft is measured by measuring the length of suspended wire, or by some other suitable means.
4. The centre line inside the tunnel should be maintained by a precise theodolite (Wild T-2 theodolite, for example) provided with an artificial illumination system to enable work at night and in the darkness of the tunnel.
5. Care should be taken that the centre line maintained from both ends and the one transferred from top coincide.

E. Preparation of Drawings

The following drawings have to be prepared for a tunnelling project:

1. A detailed drawing showing the centre line of the tunnel on the top-hill surface with proper notation on the pegs.
2. A detailed drawing showing the position of the shaft, depth of shaft, etc.
3. A detailed drawing of the tunnel section, formation level, etc.

12.11 TOPOGRAPHIC SURVEY

A. Introduction

Topographic survey involves determining the horizontal and vertical locations of objects on the surface of the earth.

Horizontal location entails locating 'objects' like roads, railways, ponds, houses, boundaries of properties, etc. by measuring horizontal distances; the objects are indicated by symbols.

Vertical location includes the location of hills, valleys, depressions, benchmarks, RLs of points, etc. by measuring vertical distances; the objects in this case are represented in relief.

Thus, a topographic map shows the nature of the earth surface along with the positions of different objects.

Such a map is essential for the engineering projects involving roads, railways, irrigation, reservoirs, townships etc. The scale of a topographic map depends on the extent of area it covers, and the purpose for which it is to be prepared. Generally topographic map is prepared according to a scale of 1 cm to 1 km (i.e. 1/100,000).

B. Representation of Relief

From the general map of a country, only the positions of objects can be found out, and the nature of the ground surface (i.e. the location of undulations, hills, valleys, depressions etc.) cannot be identified.

To indicate the nature of the ground, the map is represented in relief. The following are the different methods generally employed for representation of relief.

1. By spot level (reduced level) In this system, the RLs of specific points are found out and noted in the map (Fig. 12.6).

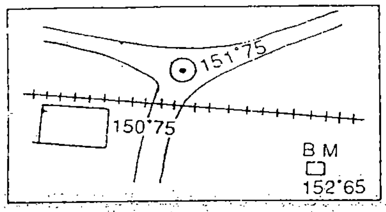
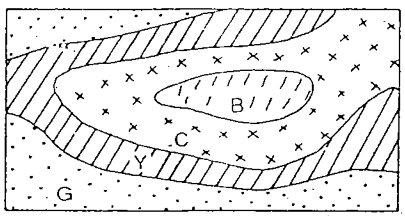


Fig. 12.6

2. By colouring In this system, a certain colour convention is assumed for a particular range of altitudes (e.g. green for RL 150–152, yellow for RL 152–154, crimson for RL 154–156, and so on) and each zone is shown in light colour wash. This system of relief is mainly adopted in geographical maps (Fig. 12.7).



G=GREEN Y=YELLOW
C=CRIMSON B=BLUE

Fig. 12.7

3. By shading Considering rays of light to fall vertically on the ground surface, light intensity would differ according to the configuration of the ground. Flat ground and ridge lines appear white, but the slope of the ground appears shaded. The shading is dark on steep slopes and light on gentle slopes (Fig. 12.8).

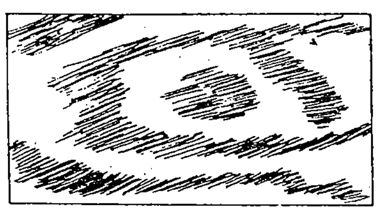


Fig. 12.8

4. By hachures

(a) Vertical Hachures This system is generally employed to show hills or depressions where the contours are closed. In this system parallel lines are drawn in the direction of flow (i.e. perpendicular to the contour line). On steep slopes, the lines are closer and thicker. On flatter slopes, they are wider and finer (Fig. 12.9).

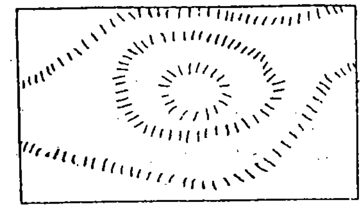


Fig. 12.9

(b) Horizontal Hachures In this system short lines (like small dashes) are drawn along the contour lines. On steep slopes, the lines are thicker and on flatter slopes, they are thinner (Fig. 12.10).

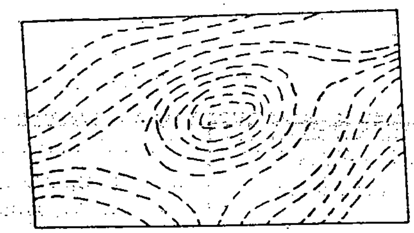


Fig. 12.10

(c) By Contour Lines In this system the nature of the ground surface is shown by contour lines (Fig. 12.11).

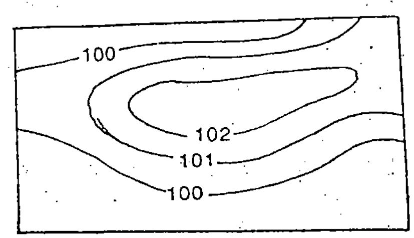


Fig. 12.11

C. Procedure of topographic surveying

1. Triangulation or Traversing In this method, the area to be surveyed is first enclosed by horizontal control and vertical control points (i.e. main stations). Horizontal control represents the relative horizontal positions of points and vertical control the relative altitudes of these points. Then, triangulation or traversing is done on the basis of the natural features of the area.

It the triangulation method, the whole area is divided into well-conditioned triangles, the sides of which are measured accurately by invar tape or computed by tacheometer. The magnetic bearing of the base line of triangulation is accurately measured by theodolite. The angles between main survey lines adjacent to each other are fixed by chain angles.

In the method of traversing the whole area is enclosed by "closed traversing". The magnetic bearing of the base line (the line should be long enough and pass through fairly level ground) is accurately measured by theodolite, as are the interior angles of the traverse. The lengths of traverse legs are measured accurately by invar tape, or by theodolite using the stadia method.

If the area is extensive, it is divided into a number of sectors. Each sector is enclosed by a closed traverse having proper connection with the other sectors (i.e., it is ensured sectors have that the common sides).

2. Location of objects In case of triangulation survey, the objects are located by taking offsets with respect to the survey lines. The offsets are noted in a field book, maintaining a proper sequence of survey lines (i.e. AB, BC, CD, ...) Then a map is prepared by plotting field records according to a suitable scale.

In case of theodolite traversing, first the traverse is plotted to a suitable scale by the coordinate method (i.e. on the basis of latitude and departure), and then the objects are located on the map by plane table by radial and intersection methods. If the objects are located by the transit-and-tape method, a field book has to be entered while measuring the traverse legs. In this case, plotting is done afterwards.

3. Location of contour During the process of locating objects, the contours are located on the map. This may be done directly by plane table. However, this method is very laborious.

The contours may also be located indirectly by dividing the area into squares or by taking cross-sections. Spot levels are taken on the relevant points by means of a levelling instrument. Then the RLs of the corners of the squares or specific points on the cross-sections are noted on the map, and the contour lines are drawn by interpolation.

Also, rather than by drawing contour lines, the nature of the ground may be indicated in relief (as described previously).

12.12 PROJECT ON TOWNSHIP OR CITY SURVEYING

A. Introduction

A project on township or city surveying involves proper coordination of all types of development work necessary for the township or city. It entails implementation of the following systems:

1. Street system
2. Property lines
3. Water supply system
4. Sanitary system
5. Electrification system
6. Telephone system

In the case of a newly developing township or the modification of one in existence for some time, a plan for development should be clearly worked out so that the facilities provided in the master plan may be suitably extended.

A project survey for the township should be conducted in order to prepare maps in respect of the following:

1. Topography
2. Street survey
3. Property demarcation
4. Water supply
5. Sanitary system
6. Electrification system
7. Telephone system

The procedure for preparing these maps are discribed later.

All the surveys are conducted by the development authority of the city or town, in coordination with concerned departments like the Roads Department, Municipality or Corporation, Electric Supply Department, Telephone Department, etc.

B. Instruments Required for Conducting City Survey

The following instruments are required for conducting a city survey:

1. Transit theodolite with stand
2. Levelling instrument with stand
3. Levelling staff
4. Metallic tape
5. Invar tape
6. Metric chain with arrows
7. Plane table with accessories
8. Ranging rods, optical square, pegs, etc.

C. Preparation of Topographic Map

The entire area is divided into a number of sectors, each of which is enclosed by a polygon. The polygons are connected by common sides. They are treated as closed traverse and the traversing is done by theodolite. The interior details are located by plane table or the transit- and tape-method. These details include houses, roads, lakes, parks, railway lines, stations etc.

Fly levelling is undertaken to establish the RLs of important points and benchmarks. Contouring is done by plane table or tacheometer. The nature of the ground surface is indicated in relief (i.e. colouring, shading, hachuring, etc.) in the map.

Finally, all the sectors are assembled in one map so that the township area or city area may be seen at a glance. (Fig. 12.12).

D. Preparation of Street Map

The street map is prepared using a large scale to show distinctly the network of streets, roads, lanes, parks, etc. Names of streets, roads and parks should be mentioned. In this case also the township area is divided into different sectors. The street map of each sector is prepared by plane table. In such a map the location of interior details (i.e. lakes, houses, and other properties) need not be shown. Fly levelling is done along the streets and benchmarks are established at different points for future reference (Fig. 12.13).

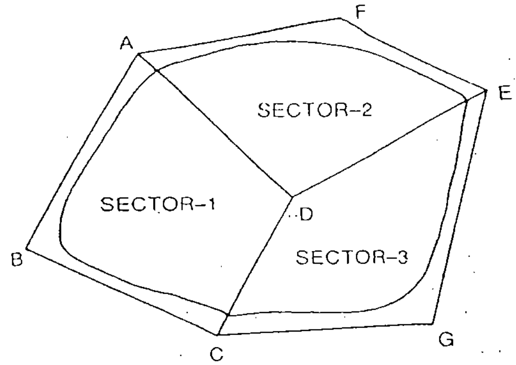


Fig. 12.12

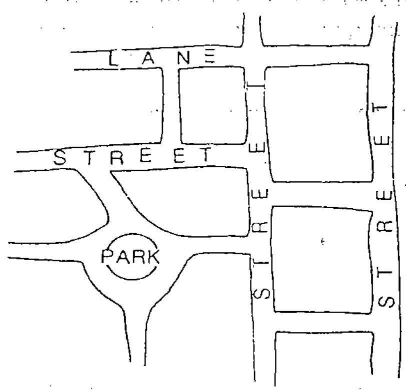


Fig. 12.13

E. Preparation of Property Map

This map is also prepared using a large scale to show the boundaries of public and private properties, plot numbers, premises numbers, and so on. The property map is also prepared by plane table by dividing the total township area into different sectors (Fig. 12.14).

F. Preparation of Water Supply and Sanitary Map

The network of the water supply distribution system and that of sewer lines are laid underground. So, to facilitate location of any spot from top, the entire networks of the water supply and sanitary works are shown by conventional lines on the street survey map. All the essential lines and points (such as intake point, purification point, check valves, fire hydrants, manholes, lampholes, inspection chambers, etc.) are marked with specific symbols.

G. Preparation of Electrification Map

Generally, the network of cables for electrification of the township or city is also

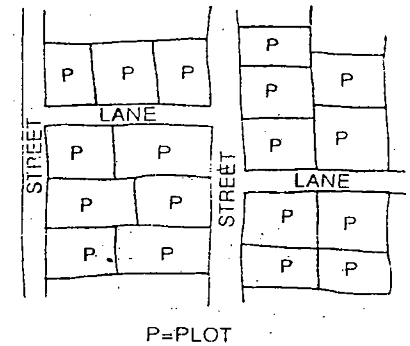


Fig. 12.14

laid underground. Therefore, the methods of distribution should be marked on the street survey map using a different colour convention or by suitable symbols. The specific points of the supply line should be clearly marked to aid easy location of any spot incase of cable fault. The network of the supply system should be indicated on the street survey map even if the electrification is through overhead lines.

H. Preparation of Telephone Map

The network of telephone cables also lies under the ground. So, to aid location of a fault, the network system is shown on the street survey map using a certain colour convention or by suitable symbols.

I. Coordination Work

The development authority of the city or town is responsible for the construction of streets, roads, culverts, bridges, flyovers, bypasses, subways, parks, public places, etc.

The sanitary and water supply system is implemented by the municipality or corporation. The electrification programme is carried out by the state electricity board, and the telecommunication system is run by the telegraph department.

13

Hydrographic Survey

13.1 INTRODUCTION

Hydrographic survey includes all types of hydrological observations which are necessary for the design of hydraulic, or marine structures. For the design of dams, barrages and weirs, and for cross-drainage works, a knowledge of the amount of rainfall in the catchment area and the discharge through the rivers are essential. Again for the design of ports, docks, harbours, light houses, etc. and location of hills and sand bars under the sea, shore line and tidal survey, and echo-sounding are extremely important. Hydrographic survey includes the following:

1. Rain gauging
2. River gauging
3. Marine survey

13.2 RAIN GAUGING

A. Necessity

The amount of rainfall in the catchment area of a basin or a river is generally recorded. This helps to predict the probable run-off through the basin or river. Thus, any probable flood in the downstream area may be forecast in case there is heavy rain, and necessary precautions may be taken to avert disaster.

B. Location of Site for Rain Gauge Station

The recommended network of rain gauge stations depends on the topography of the catchment area. Generally, one rain gauge for every 500 km^2 for plane areas and one for every 150 km^2 for hilly areas i.e. considered appropriate. To get the proper rainfall records from the catchment area, the stations should be so selected that they may fully cover the basin. The following points should be remembered while selecting rain gauge stations:

1. They should be established on level ground and not on sloping ground
2. They should not be close to any permanent structure or tree
3. They should be protected from high winds
4. They should be accessible

C. Measurement of Rainfall

The rainfall is recorded by different types of rain gauges, such as the non-recording type (Symon's rain gauge) and the recording type or automatic rain gauge.

The construction and operation of Symon's rain gauge are described below.

It consists of a metal cylindrical casing of diameter 127 mm and height 305 mm. It is secured in a concrete foundation. The cylinder consists of a brass rim on the top. A glass receiving bottle of diameter 100 mm and height 203 mm is inserted into the casing. A funnel of diameter 127 mm is placed on the glass bottle so that the top of the funnel is just flush with the brass rim. Thus the rain water is allowed to collect directly in the glass bottle. A graduated measuring glass cylinder is provided with each rain gauge for measuring rainfall to the extent of 0.2 mm. Normally, a reading is taken once in 24 hours at 8.30 a.m. every day. But on days when there is heavy rainfall, readings are taken three or four times in a day, because the receiving glass bottle may overflow. A register is maintained for recording the rainfall and reports are sent to the concerned office for necessary action (Fig. 13.1).

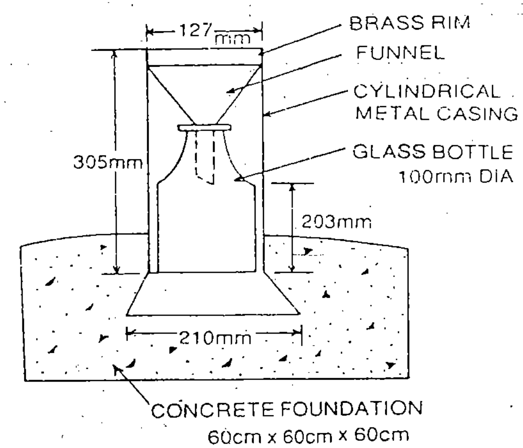


Fig. 13.1

13.3 RIVER GAUGING

A. Necessity

River gauging involves the measurement of discharge of a river and the establishment of a gauge post on one of its banks. This is done to directly read the HFL so that a warning may be given to surrounding areas for any precautionary measures to be taken if necessary. In addition, river gauging is undertaken for the following reasons.

1. To fix the number of spans of road and railway bridges, so that unexpected high floods may not cause any damage to the structure
2. To fix the height of the guide bank

3. To fix the spillway level and the height of the dam or barrage
4. To design the cross-drainage works when a canal crosses a river

B. Selection of Discharge Site

The following points should be remembered while selecting a discharge site:

1. The river should be straight for a minimum length of about 500 m or four times its width (Fig. 13.2).

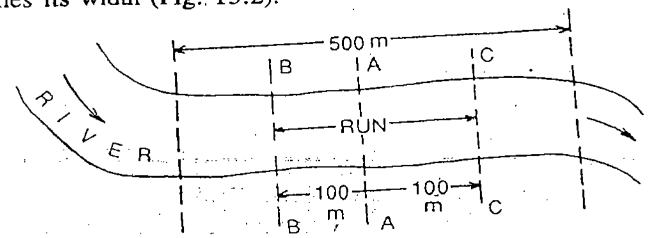


Fig. 13.2

2. The section of the discharge site should be well-defined (Fig. 13.3).

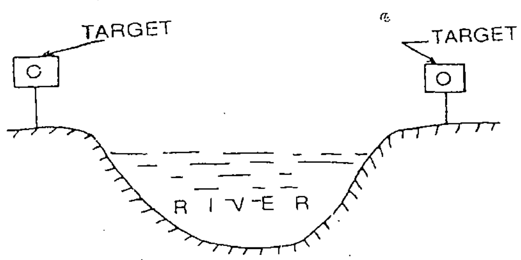


Fig. 13.3

3. The flow should be confined to a single channel. Bifurcation is not allowed (for instance, in the river shown in Fig. 13.4, a section through N–N is not allowed).

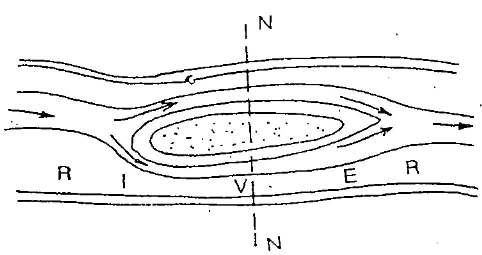


Fig. 13.4

4. The centre line AA of the discharge site is marked with targets on both banks (Fig. 13.3).

Two sections BB and CC are marked on the upstream and downstream sides of the discharge site at distances of 100 m. The distance between BB and CC is known as **'Run'** (Fig. 13.2).

C. Fixing the Gauge Post

Gauge posts are wooden posts of section 10 cm × 5 cm and length 2 m, graduated in metres and fractions of metres so that a minimum reading of up to 0.01 m can be taken. The posts are fixed in series on a concrete foundation along the slope of the river bank. The bottom post is assumed to be at zero and the other posts are graduated accordingly (Fig. 13.5).

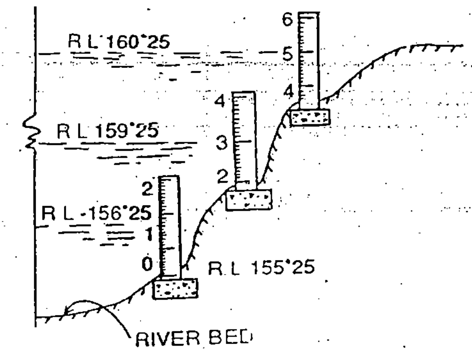


Fig. 13.5

The RL of the zero mark of the starting gauge post is determined by fly levelling from a benchmark. Then by simply noting the reading on the gauge, the RL of the water level can be predicted.

For example, suppose the RL of the zero mark is 155.25 m. So, when the gauge reading is 1 m, the water level is at RL 156.25 m, when the reading is 3 m, the RL of the water level is 158.25 m, and so on.

Measurement of Depth of Water (Sounding)

Depth of water is measured by the following methods:

1. Sounding rod It is a wooden or bamboo pole of 5 cm diameter and 2.5 m length. It is provided with a disc of 15 cm diameter at the bottom. It is graduated in metres and 1/10th of a metre. The sounding rod is suitable for measuring the depths of small rivers, of the order of 2 m. To measure depth, the sounding rod is slowly immersed vertically from a boat, at the required place, so that the base plate just touches the bed of the river. The depth of water is then noted from the graduation (Fig. 13.6).

2. Sounding cable When the depth of water is more than 2 m and water current is high, the velocity rod is not practicable. In such a case, a cable or rope is

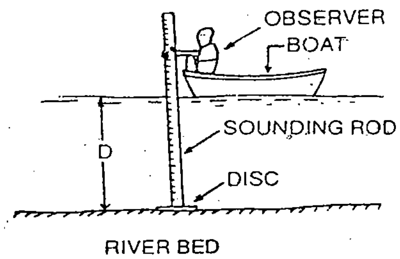


Fig. 13.6

released slowly from a boat by means of a pulley system (Fig. 13.7). The cable consists of a counter weight of about 5 kg. When the weight just touches the bottom of the river, a mark is made on the cable exactly at the water level. Then the depth is measured by tape.

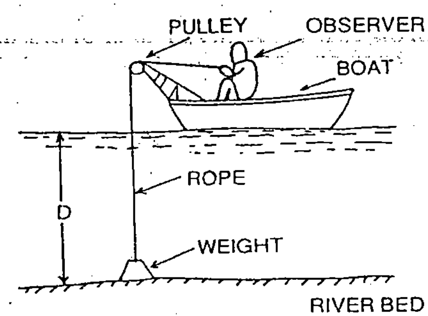


Fig. 13.7

3. Echo-sounder The echo-sounder is used for measuring depths of large rivers, in excess of 10 m, and those of seas. It is an electrical instrument, in which a sound impulse from the surface of water is sent towards the bottom of the river or sea (Fig. 13.8). The sound waves are reflected back from the bed in the form of an echo which is arrested by the receiver. There is an automatic recording of the time of onward and backward travel of the sound wave. Taking the velocity of sound in water to be approximately, 1,470 m/s, the depth of water can be computed. Generally, the echo-sounder is placed just at the water level to eliminate transmission loss and obtain an accurate result.

E. Measurement of Velocity of Flow

Velocity of flow can be measured by the following methods:

1. The surface float method
2. The sub-surface float method
3. The velocity rod method

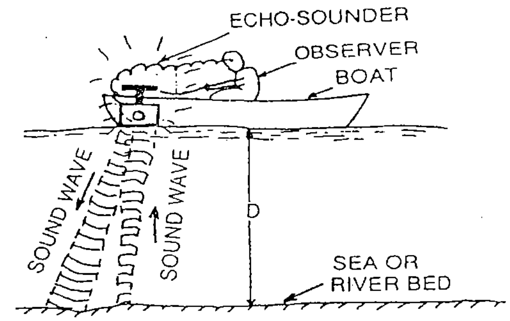


Fig. 13.8

4. The Pitot tube, method
5. The current meter method

1. The surface float method Surface floats are made of cork, and can easily float on water. They are generally in the form of 10 cm cubes. The floats are painted red or white and have a small flag attached on top (Fig. 13.9(a)).

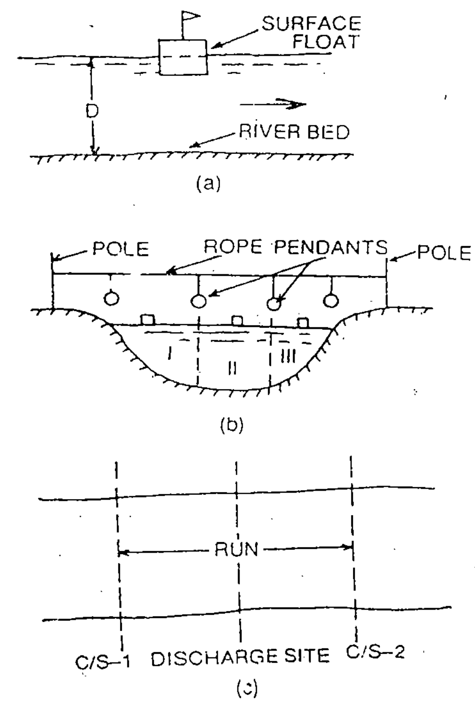


Fig. 13.9

Velocity is measured in the following way:

(a) At the discharge site a rope is stretched between two poles fixed on both banks of the river and the water section is divided into several compartments by hanging tags from a rope (Fig. 13.9(b)).
(b) The velocity of each compartment is measured by the float, which is released slightly ahead of c/s 1. When the float just crosses the section, the stopwatch is started. When it just crosses c/s 2, the stopwatch is closed. The time taken by the float to cover the 'run' (known distance) is noted, and from this the velocity is calculated (Fig. 13.9(c)).
(c) Similarly, the velocities of all the compartments are measured and the average of these is taken as the mean velocity. But this velocity represents only surface velocity and is not the actual mean velocity of the river.

2. The sub-surface float method The sub-surface float is a hollow cylinder which is attached by a cord to a surface float. The position of the sub-surface float is adjusted according to the depth of the river. Generally, it is kept 0.2*D* metres above the bottom. In this case, the measured velocity is equal to the mean velocity. The procedure of measurement is the same as that in the surface float method (Fig. 13.10). Here *D* is the depth of water.

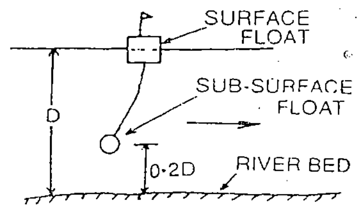


Fig. 13.10

3. The velocity rod method The velocity rod is made of hollow metal or wood. Its diameter is about 2.5 to 5 cm. A weight is provided at the bottom of the rod to keep it vertical. The length of the rod is adjustable. The rod is generally submerged to the extent of 0.6*D*. The procedure of measurement is exactly similar to that in the surface float method (Fig. 13.11).

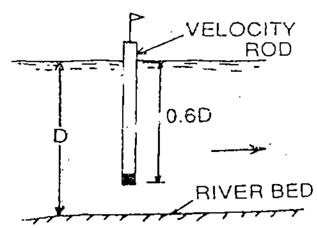


Fig. 13.11

4. The Pitot tube method The Pitot tube is a glass tube bent at an angle of 90° at the lower end. The upper end is open and kept above the water level. The lower end is in the form of a nozzle, and is directed upstream. Due to the velocity of water, the water column rises in the right limb (Fig. 13.12). Let the rise in water level be *h*. Then applying Bernoulli's theorem between points A and B,

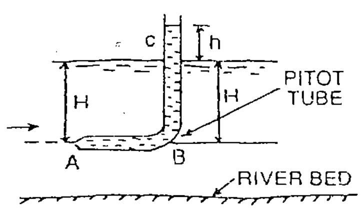


Fig. 13.12

$$H + \frac{v^2}{2g} = H + h$$

or

$$V = \sqrt{2gh}$$

So, by measuring the rise of water column the velocity can be calculated. The Pitot tube is held along the centre line of the discharge site at the midpoint of every compartment.

3. The current meter method The price current meter (Fig. 13.13) is commonly used for measuring the velocity of water. It consists of the following components.

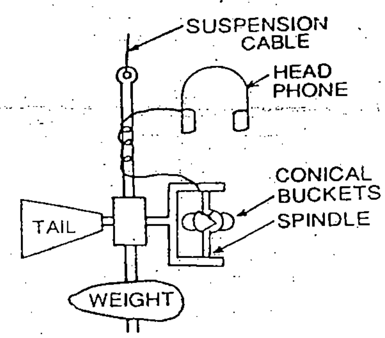


Fig. 13.13

(a) A tail which keeps the meter in the direction of alignment.
(b) A system of conical buckets fitted to a spindle which rotates due to current.
(c) A counterweight which keeps the meter vertical.
(d) A headphone for registering the sound produced by the spindle at every revolution

The current meter is lowered to a depth of 0.6*D* (*D* being the depth of water) by a suspension cable. The system of conical buckets is kept on the upstream side and the tail on the downstream side. Due to the velocity of water, the system of conical buckets rotates about the spindle and a 'tick' sound is emitted once for every revolution. Thus, a continuous "tick-tick" sound is heard through the headphone. The number of revolutions per minute is counted with the help of a stopwatch. The velocity can be ascertained from the rating table (supplied by the manufacturer with the meter). This velocity gives the mean velocity of flow. The current meter is immersed along the centre line of the discharge site at the midpoint of every compartment.

F. Determination of Cross-Sectional Area of a River

Case 1—When the river is small Two poles are fixed on both banks of the river and a rope is stretched between them (Fig. 13.14). Then the water-width of the river is divided into several compartments of equal width (say *b*) and the points are marked by hanging tags or pendants. The depth of water corresponding to each pendant is measured by sounding rod or sounding cable. Let the depths of

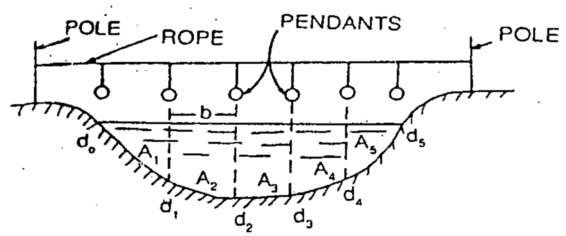


Fig. 13.14

water be d_0, d_1, d_2, ... Considering the end compartments as triangles and other compartments as trapeziums, the area is calculated as follows.

$$A_1 = \frac{1}{2} d_1 \times b \qquad A_2 = \frac{d_1 + d_2}{2} \times b$$

$$A_3 = \frac{d_2 + d_3}{2} \times b \quad \text{and so on.}$$

$\therefore$ Total cross-sectional area $= A_1 + A_2 + A_3 + \ldots$

Case II—When the river is large The centre line AB (Fig. 13.15) is taken perpendicular to the river. Two stout pegs T_1 and T_2 are fixed on AB. Then the following procedure is adopted.

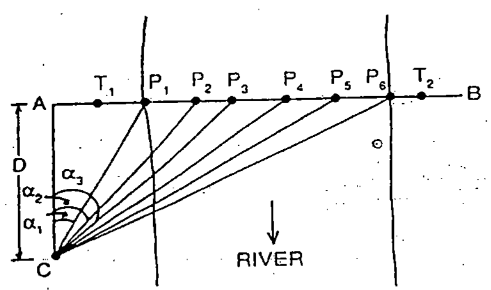


Fig. 13.15

1. At station A, a theodolite is set up and the line is ranged properly. The width of the river is measured by stadia method.
2. The width is divided into equal parts, and the widths of these compartments are fixed. Thus the distances AP_1, P_1P_2, P_2, P_3, ... are fixed.
3. Another theodolite is set up at C, so that AC is perpendicular to AB. The distance AC is measured (say it is equal to D).
4. Then the angles α_1, α_2, ... are calculated, in the following manner.

$$\alpha_1 = \tan^{-1} \frac{AP_1}{AC}, \qquad \alpha_2 = \tan^{-1} \frac{AP_2}{AC} \qquad \text{and so on.}$$

5. The calculated angles α_1, α_2, ... are set out from the theodolite at C. Thus the line of sight of the theodolite at A and that of the theodolite at C coincide at the points P_1, P_2 P_3, ... which are the points along the cross-section where soundings are to be taken.
6. The depths of water at these points are measured by a sounding cable or echo-sounder mounted on a boat.
7. Then the cross-sectional area of the water section is calculated in the usual manner.

G. Calculation of Discharge of a River

Let the calculated cross-sectional area of the water section be A m^2. The mean velocity is measured by any suitable method. Let it be V m/s.

Then Discharge $Q = A \times V$ cumecs

13.4 MARINE SURVEY

Marine survey involves the following procedure:

1. The shore line at low and high tide is marked. Then an open traverse is conducted by compass or theodolite to obtain the configuration of the shore line.
2. The depth of water around the harbour area is determined by echo-sounder and obstructions like large rocks, sandbars, etc. are located.
3. Current and tidal observations are taken to identify the nature of the sea water and take necessary precautions for safe movement of ships.
4. Positions where lighthouses are to be set up for indicating a safe route for ships from one port to another are established. The positions of large rocks, submerged hills, sandbars, etc. are located by measuring depth by echosounder and other appliances.
5. The profile of the bottom of the sea is prepared by depth recording instruments mounted on a motorboat or steamer along a pre-determined alignment fixed by compass. Soundings are taken at regular intervals and recorded.
6. Maps are prepared to indicate the position of submerged hills, sandbars or any other obstructions. The safe route is indicated on the map by showing the positions of lighthouses. The profile of the sea bed is prepared for navigational guidance.

14

Setting Out Works

14.1 SETTING OUT A BUILDING

A building is set out in order to clearly define the outline of excavation and the centre lines of the walls, so that construction can be carried out exactly according to plan. The centre line method of setting out is generally preferred and adopted.

Procedure

1. From the plan, the centre lines of the walls are calculated. Then the centre lines of the rooms are set out by setting perpendiculars in the ratio 3 : 4 : 5. Suppose the corner points are A, B, C, D, E, F, and G which are marked by pegs with nails on top.

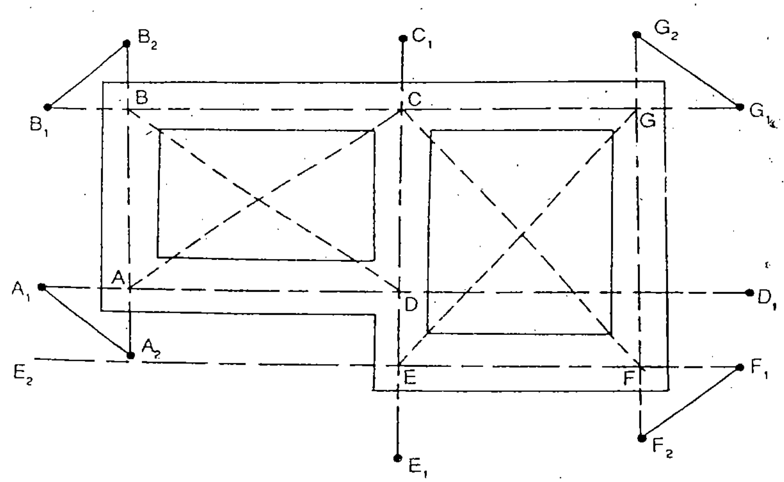


Fig. 14.1

2. The setting of the corner points is checked according to diagonals AC, BD, CF and EG.

3. During excavation, the centre points A, B, C, ... may be removed. Therefore,

the centre lines are extended and the centre points marked about 2 m away from the outer edge of excavation. Thus the points A_1, A_2, B_1, B_2, ... are marked outside the trench. The centre lines are shown clearly by stretching thread or rope. The centre points fixed 2 m away from the excavation are marked with stout pegs.

4. From the plan, the width of excavation is found out and set out around the centre line. The excavation width is also marked by thread with pegs at appropriate positions.

5. The excavation width is then marked by lime or by making a furrow with a spade.

6. If the plan is much too complicated and follows a zig-zag pattern, the centre pegs are kept at suitable positions according to site conditions.

14.2 SETTING OUT A CULVERT

The abutments and wing walls of a culvert are set out by coordinates from the centre of the culvert, which is taken as the origin.

From the plan, the coordinates of different points on the abutments and wing walls are calculated as tabulated as follows:

Point	Coordinate		Remark
	x	y	
P_1	x_1	y_1	Data calculated for left half of culvert
P_2	x_2	y_2	
P_3	x_3	y_3	
P_4	x_4	y_4	

As the culvert is symmetrical about both the axes, the necessary data for the right half are similar to those for the left half. Again, the data for the both sides of the X_1X line are similar (Fig. 14.2).

Procedure

1. The axes (centre line) XX_1 and YY_1 are set out, with O is the origin.
2. Along the line OX_1, points 1, 2, 3 and 4 are marked at distances x_1, x_2, x_3 and x_4 from origin.
3. Now Along OY, points 1′, 2′, 3′ and 4′ are maked at distances y_1, y_2, y_3, and y_4 from O.
4. Now two tapes are taken with their zero ends together and held at P_1 by the overseer in such a way that when two assistants hold them fully stretched at 1 and 1′, the readings are exactly equal to y_1 and x_1 (i.e. $P_1 1 = y_1$ and $P_1 1' = x_1$). Then the point P_1 is marked with a stout peg.
5. Similarly the other pegs are fixed at P_2, P_3 and P_4. The pegs for the right half are fixed according to the data available from the table prepared for the left half. The culvert is symmetrical about the centre line XX_1. So, the other half can be easily set up.

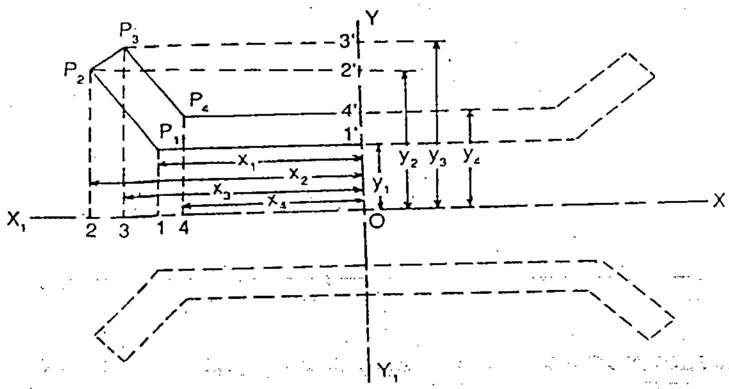


Fig. 14.2

14.3 LOCATION OF BRIDGE PIER

The procedure for locating a bridge pier is as follows:

Procedure

1. The centre line AB of the bridge is ranged by setting a theodolite at A and two stout pegs are fixed at T_1T_2 on the line AB (Fig. 14.3).

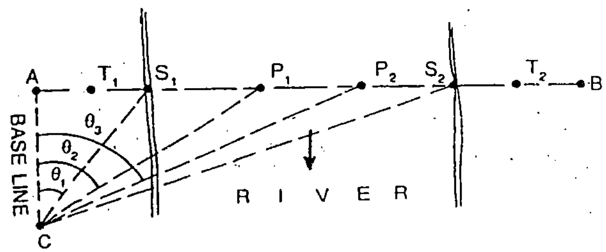


Fig. 14.3

2. The effective length of the bridge (S_1S_2) is determined by triangulation or by the stadia method.
3. The number of spans and the width of each are theoretically ascertained and plotted in a plan.
4. A base line AC is taken perpendicular to the centre line AB. The distance AC is accurately measured by invar tape.
5. On the plan, points S_1 and S_2 represent the centres of abutments and points P_1 and P_2 the centres of piers.
6. The distance AS_1 is measured accurately by invar tape.
7. From the effective width of the bridge, the widths of span S_1P_1, P_1P_2 and P_2S_2 are ascertained.
8. Then the angles θ_1, θ_2, θ_3 and θ_4 are calculated as follows.

$$\theta_1 = \tan^{-1}\frac{AS_1}{AC} \qquad \theta_2 = \tan^{-1}\frac{AP_1}{AC}$$

and so on.

9. Another theodolite is set up at C and the angles θ_1, θ_2, ... are set out in the usual manner.
10. The line of sight of the theodolite at A and that of the theodolite at C will intersect at points S_1, P_1, P_2 and S_2. These points are marked in position accurately.
11. In order to ensure extreme accuracy, the centre lines of the piers are checked by setting a theodolite on a base line taken on the other bank of the river.

15

Model Questions with Answers

At the end of every chapter, section on short questions with answers is included. In this chapter, some additional questions with answers are given for the benefit of students appearing in competitive examinations.

Q. 1 Distinguish between plane surveying and geodetic surveying.

Ans. The following are the points of distinction:

(a) In plane surveying, the curvature of the earth is *not taken into account.* But in geodetic surveying the curvature is *taken into account.*

(b) In plane surveying, *short distances and small areas* are covered. But geodetic surveying covers long distances and *large areas.*

(c) In plane surveying, the area covered is *less* than 260 km^2 or 100 sq. miles. But in geodetic surveying the area covered is *more* than 260 km^2 or 100 sq. miles.

(d) In plane surveying, the lines are considered *straight.* But in geodetic surveying the lines are assumed to be *curved.*

(e) In plane surveying, the *angles* and *triangles* are treated as *plane.* But in geodetic surveying, the angle and triangles are assumed to be *spherical.*

Q. 2 While considering the curvature of the earth, what is the difference in length between an arc and the corresponding chord?

Ans. It is found that the difference is:
10 cm in a length of 18.2 km,
30 cm in a length of 54.5 km, and
50 cm in a length of 91 km.

Q. 3 What is the difference between the sum of angles of a plane triangle and that of spherical triangle?

Ans. The difference of the sum of angles is only one second for an area of 195.5 km^2.

Q. 4 What is the advantage of triangulation ?

Ans. The advantage is that the triangles formed during this process can be easily plotted by knowing only the lengths of their sides; no angular measurements are required.

Q. 5 In chain surveying, how can the direction of adjacent sides be fixed?

Ans. The direction can be fixed by forming chain angles with the help of tie stations and tie lines.

Q. 6 What are the different types of survey?

Ans. (a) *Topographical survey* This is done to determine the locations of towns, villages, roads, railways, rivers, etc. and to know the nature of the ground surface.

(b) *Cadastral survey* Cadastral survey is undertaken to mark the boundaries of fields and other properties.

(c) *Geographical Survey* This is done to locate hills, valleys, forests and other geographical features of a country.

(d) *Geological survey* Geological survey is conducted in order to identify the different strata of the crust of the earth and expose concealed wealths like oil, natural gas, etc.

(e) *Archeological survey* This is done to unearth ancient relics.

(f) *Mine survey* The aim of such survey is to identify areas for exploring mineral wealth like coal, iron ore, mica, gold, etc.

(g) *Military survey* This is done to set up road and railway communication for the defence of a country.

(h) *City survey* City survey is undertaken to show the different plots, streets, sewer lines, water lines, etc. in a city.

(i) *Contour map* Such a map is prepared in order to illustrate the nature of the ground surface in a particular area.

(j) *Engineering survey* This is done for the projects involving roads, railways, irrigation canals, etc.

(k) *Hydrographic survey* This is undertaken in order to obtain rainfall records in a catchment area and measure the discharge of a river (river gauging).

(l) *Marine survey* This is done for fixing the positions of docks, harbours, lighthouses, etc.

Q. 7 What are the different types of scales?

Ans. (a) *Plain scale* Using this scale two consecutive units or 1/10th of a unit can be measured.

(b) *Diagonal scale* With this scale three consecutive units, or 1/100th of a unit, can be measured.

(c) *Comparative scale* Using such a scale, two different systems of units can be measured, e.g. cm v. inch, metre v. feet, time v. distance, etc.

(d) *Vernier scale* The vernier scale is meant for measuring fractional parts of the smallest division of the main scale.

(e) *Direct vernier* Here, the vernier division is shorter than the primary division.

(f) *Retrograde vernier* In this scale, vernier division is longer than the primary division.

(g) *Double vernier* Here, vernier graduations are marked in both directions.

Q. 8 What would you mean by the terms 'small' and 'large' in the context of a scale?

Ans. In a small scale, 1 cm represents a large distance, e.g. 1 cm = 1 km. In a large scale, 1 cm represents a small distance, e.g. 1 cm = 1 m.

Q. 9 What is pacing?

Ans. Pacing involves measurement of distance by counting the number of paces or steps. The walking step of a man is considered to be equal to 80 cm or 2.5 ft.

Q. 10 What is a test gauge?

Ans. It is a standard length fixed by pegs on a level platform to standardise a chain.

Q. 11 What is invar tape?

Ans. It is a tape made of an alloy of 64% steel and 36% nickel, and used for high-precision work.

Q. 12 What are the different types of minor instruments? Mention their uses.

Ans. (a) Cross-staff—for taking perpendicular offsets
(b) Optical square—for taking perpendicular offsets
(c) Clinometer—for measuring angles of slopes
(d) Box sextant—for measuring any horizontal angle
(e) Abney level—for measuring angles of slopes and setting out grades
(f) Planimeter—for measuring area on the map
(g) Pentagraph—for enlarging or reducing area of the map
(h) Ceylon ghat tracer—for measuring angles of slopes and setting out grade contours
(i) Subtense bar—for measuring horizontal angles in stadia method
(j) Range finder or line ranger—for ranging a chain line
(k) Hand level—for marking contours, and cross-sections in preliminary work

Q. 13 What types of clinometers do you know of?

Ans. (a) De Lisle's clinometer
(b) Watkin's mirror clinometer
(c) Foot rule clinometer
(d) Indian pattern clinometer

Q. 14 What are the expressions required in tape correction?

Ans. (a) Absolute correction, $C_a = \dfrac{L \times C}{l}$

(b) Temp. Correction, $C_t = \alpha (T_m - T)\, l$

(c) Pull correction, $C_p = \dfrac{(P_m - P_o)\, l}{AE}$

(d) Sag correction, $C_s = \dfrac{l w^2}{24n^2 \times P_m^2}$

(e) Hypotenusal allowance, $C_h = 1 (\sec \theta - 1)$

Q. 15 What are the various ways of expressing the amount of slope correction?

Ans. The correction may be expressed in the following forms.

(a) $C_h = \dfrac{h^2}{2 - l}$

(b) $C_h = l\,(1 - \cos \theta)$

(c) $C_h = l \text{ versin } \theta$

(d) $C_h = 2l \sin^2 \dfrac{\theta}{2}$

(e) $C_h = 0.00015\, l\, \theta^2$

(f) $C_h = \dfrac{l}{n^2}$ (slope 1 in n)

Q. 16 How are compensating and cumulative errors related to the length of a line?

Ans. Compensating error $\alpha \sqrt{L}$
Cumulative error $\alpha\, L$

Q. 17 What is the error due to incorrect ranging?

Ans. Error in length $= \dfrac{d^2}{40}$ (for 20 m chain) $= \dfrac{d^2}{60}$ (for 30 m chain)

where d is the amount of deviation.

Q. 18 What is the permissible error in chaining?

Ans. For ordinary work—1 in 1,000
For precise work—1 in 2,000

Q. 19 What is the angle between the mirrors in an optical square?

Ans. The angle is 45°.

Q. 20 On what measurements are chain survey and theodolite survey based?

Ans. Chain survey is based on linear measurements, and theodolite survey on both linear and angular measurements.

Q. 21 What is the fundamental difference between a prismatic compass and a surveyor's compass?

Ans. A prismatic compass gives whole circle bearings from 0° to 360°, whereas a surveyor's compass gives quadrantal bearings from 0° to 90°, with notations NE, NW, SE and SW.

Q. 22 What is the amount of dip at the equator?

Ans. The amount of dip at the equator is zero.

Q. 23 What are isogonic and agonic lines?

Ans. An isogonic line is one passing through points of equal declination, whereas an agonic line connects points of zero declination.

Q. 24 What is secular variation?

Ans. The variation of magnetic declination after every 100 years, similar to the oscillation of a pendulum, is known as secular variation.

Q. 25 What is the limit of precision in traversing?

Ans. The angular error of closure should not exceed $15' \times \sqrt{N}$ (N being the number of traverse legs) and the linear closing error should not exceed 1 in 600.

Q. 26 What is an azimuth?
Ans. The true bearing of a line is known as the azimuth of that line.
Q. 27 What is a trunnion axis?
Ans. The horizontal axis about which a telescope can be rotated in a vertical plane is known as a trunnion axis.
Q. 28 What is the difference between a magnetic meridian and a true meridian?
Ans. A magnetic meridian represents the line indicated by a freely suspended magnetic needle.

At true meridian represents the line passing through the geographical north and south poles.
Q. 29 What is the nature of true meridians at different places?
Ans. True meridians are not parallel at different places. They converge from the equator towards the pole.
Q. 30. What are grid lines?
Ans. The assumed meridians parallel to the true meridian at a place are known as grid lines.
Q. 31 What types of telescope do you know of?
Ans. (a) The external focussing telescope—here the object glass moves to and fro.
(b) The internal focussing telescope—here a double concave lens is geared with the focussing screw by means of a rack-and-pinion arrangement which moves to and fro with the turning motion of the screw.
Q. 32 On which factors does the sensitiveness of a bubble depend?
Ans. The sensitiveness of a bubble depends on the viscosity and surface tension of the liquid.
Q. 33 What is the angular value per division of deflection?
Ans. Angular value, $\alpha' = \frac{d}{R} \times 206{,}265$ seconds

where d = length of one division and R = radius of curvature.
Q. 34 What does the term, magnification power, mean in the context of a telescope?
Ans. The ratio of the focal length of the object glass to that of the eye-piece is known as magnifying power. That is,

$$MP = \frac{f}{f'}$$

where f = focal length of object glass
and f' = focal length of eye-piece
This is expressed in terms of diameter, e.g. 20 dia, 30 dia, etc.
Q. 35 Give a practical example of a level surface.
Ans. The water surface of still a lake is one such example.
Q. 36 How will you distinguish between a ridge line and a valley line?
Ans. When higher values are inside the loop, it indicates a ridge line; when lower values are inside the loop, it indicates a valley line.
Q. 37 What is the principle to be followed while drawing rays by alidade?

Ans. The fiducial edge of the alidade should always be on the left of the station pin. This is the most comfortable position.
Q. 38 How will you locate an object found to be missing after a long period of time, in plane table surveying?
Ans. By working out a two-point or three-point problem, a station is established and then the missing object is located with respect to the new station.
Q. 39 What is the significance of the terms f/i and $(f + d)$ in tacheometry?
Ans. f/i is known as the multiplying constant, and has a value of 100. $(f + d)$ is said to be the additive constant, and is equal to zero.
Q. 40 What are the expressions for horizontal distance when the line of sight is (a) horizontal, and (b) inclined?
Ans. When the line of sight is horizontal,

$$\text{Horizontal distance} = \frac{f}{i} \times S + (f + d)$$

When the line of sight is inclined,

$$\text{Horizontal distance} = \frac{f}{i} \times S\cos^2\theta + (f + d)\cos\theta$$

Q. 41 How can a curve be designated?
Ans. A curve may be designated by radius or degree of curve.
Q. 42 What are the expressions required in setting out a circular curve?
Ans. (a) Tangent length $= 2R\tan\frac{\phi}{2}$

(b) Curve length $= \frac{\pi R\phi^\circ}{180^\circ}$ or $\frac{30\phi}{D}$

(c) Long chord $= 2R\sin\frac{\phi}{2}$

(d) Apex distance $= R\left(\sec\frac{\phi}{2} - 1\right)$

(e) Versed sine $= R\left(1 - \cos\frac{\phi}{2}\right)$

(f) Deflection angle $= \frac{1{,}718.9 \times C}{R}$ mins $= \frac{C \times D}{60}$ degrees
Q. 43 What is the expression for superelevation?
Ans. Super elevation, $h = \frac{BV^2}{gR}$ (for roads)

$= \frac{GV^2}{gR}$ (for railway tracks)
Q. 44 What is the expression for centrifugal ratio (CR)? What are the allowable CRs for roads and railways?
Ans. $CR = \frac{V^2}{gR}$

CR for roads $= \frac{1}{4}$ CR for railways $= \frac{1}{8}$

Q. 45 What is the relation between shift and transition curve?

Ans. The shift bisects the transition curve and the transition curve bisects the shift.

Q. 46 What is the ideal transition curve?

Ans. The clothoid spiral is the ideal transition curve.

Q. 47 What is the ideal vertical curve?

Ans. A parabolic curve is an ideal vertical curve.

Q. 48 Give a practical example of Bernoulli's lemniscate curve.

Ans. The curve formed by turning of an automobile is an example of Bernoulli's lemniscate curve.

Q. 49 What is the main consideration in designing a vertical curve?

Ans. Sight distance is the main factor to be considered in the design of a vertical curve.

Q. 50 What is the type of vertical curve provided in railways?

Ans. No vertical curve is provided in railways. Tunnelling is done where required.

Appendix A

West Bengal State Council Examinations

FIELD SURVEYING I, 1991

Group A

Q. 1 What is reconnaissance? Explain clearly why well-conditioned triangles are preferred to ill-conditioned ones. (Ans. See Secs. 2.2 and 2.3)

The sides of a triangle are 20.8, 28.6 and 41.3 m respectively. Examine whether the triangle is well-conditioned. (Ans. See Sec. 1.25)

Explain with neat sketches the principle and uses of an optical square. (Ans. See Sec. 2.12)

Q. 2 What factors should be considered while deciding the station points in the chain survey of a plot of land. (Ans. See Sec. 2.5)

What is meant by:

(i) The scale of a map or drawing, and

(ii) The representative fraction of a scale? (Ans. See Sec. 1.26)

A survey line ABC crosses a pond, B and C being on near and the opposite sides respectively. Two chain lines BX and BY are considered by the sides of the pond such that X, C and Y are on a straight line. On measurement it is found that BX = 67 m, BY = 111 m, CX = 42 m and CY = 78 m. Find the chainage of C if that of B is 329 m.

(Ans. 391.88 m)

Q. 3 Explain the terms: (i) check line, (ii) tie line, and (iii) maximum length of offset. (Ans. See Sec. 2.4)

Rule out a page of your answer book in the form of a field book and enter in it the following field readings obtained while chaining from station D to E:

(a) The chain line DE crosses a straight road almost perpendicularly, the points of intersection being chainages 5.6 and 18.1 m respectively.

(b) The perpendicular distance of a light post, to the right of the chain line, is 3.2 m, the root of the perpendicular being at chainage 18.8 m.

(c) The perpendicular distance of a tree to the left of the chain line is 4.6 m at chainage 20.3 m.

(d) Oblique offsets of a telegraph post to the right of the chain line are 11.4 m and 10.5 m respectively from chainages 13.0 m and 23.0 m.

(e) The length of the chain line DE is 28.4 m. (Ans. See Sec. 2.8)

Q. 4 Two stations A and B are not intervisible due to rising ground between them. Explain with sketches how the chain line AB can be properly ranged when both the stations are visible from some intermediate points. (Ans. See Sec. 1.8)

The following slope distances were measured along a chain line with a 20 m steel tape. It was noted afterwards that the tape was 3 cm too long. Find the true total horizontal distance.

Slope distance (m) = 18.7–13.4–10.1–16.9–11.6–17.8

Difference of elevation between ends (m) = 0.85–3.90–3.25–2.75–3.10–1.80

(Ans. 86.7499 m)

Q. 5 A steel tape was exactly 20 m long at 20°C when supported throughout its length under a pull of 20 kg. A line was measured with the tape under a pull of 10 kg and a mean temperature of 13°C, and found to be 480 m long. The cross-sectional area of the tape = 0.03 cm^2, total weight of the tape is 0.45 kg, α for steel = 11×10^{-6} per °C and $E = 2.1 \times 10^6$ kg cm^2. Compute the true length of the line if the tape was supported during measurement: (i) at every 20 m, and (ii) at every 10 m.

(Ans. (i) 479.846 m, (ii) 479.876 m)

Group B

Q. 6 What is closing error? Explain why closing error takes place and describe how it error can be balanced graphically. (Ans. See Sec. 3.17)

Explain the following terms with sketches:

Magnetic bearing, true bearing, and arbitrary bearing. (Ans. See Sec. 3.2)

A line was drawn to a magnetic bearing of 208° on an old map when the declination was 10°W. To what bearing should it be set now if the magnetic declination is 4°E? (Ans. 194°)

Q. 7 The following consecutive readings were taken with a level instrument along a chain line at intervals of 20 m. The first reading was at a change of 260 m where the RL is known to be 9.850 m. The instrument was shifted after the third and the eighth readings. Find the RLs of the following points and draw the longitudinal section showing in it the chainage data and RLs.

2.410, 1.765, 0.650, 3.485, 2.870, 2.105, 1.865, 1.020, 0.475, 1.670, 2.290 and 3.625 m

(Ans. 10.495, 11.610, 12.225, 12.990, 13.230, 14.075, 12.880, 12.260, 10.925)

Q. 8 Explain clearly: (i) fore and back bearing, and (ii) whole circle and reduced bearing. (Ans. See 3.2)

While traversing an area with a compass, the following bearings were observed. Find the amounts of local attraction at the different stations, the correct bearings of the lines and the included angles. Also draw a sketch of the plot if AB = 45 m, BC = 45 m and CD = 30 m, and show in it all the included angles.

Line	FB	BB
AB	140°45′	318°15′
BC	216°30′	38°00′
CD	209°15′	30°15′
DE	319°45′	139°45′
EA	60°15′	240°15′

(Ans. + 2°30′ at B, + 1°0′ at C. Corrected bearings: AB—140°45′ BC—219°00′ CD—210°15′ DE—319°45′ EA—60°15′)

Q. 9 From the following readings taken while fly levelling from A to D, determine the RL of A when that of D is known to be 116.72 m.

Staff at	A	B	C	D
BS	2.63	3.47	3.38	
FS		0.87	0.35	0.24

(Ans. RL of A = 108.70 m)

On taking a field measurement of a quadrilateral plot PQRS it was found that PQ = 32 m, QR = 48 m, RS = 40 m, SP = 24 m and SQ = 40 m. Find the area of the plot. Also find graphically or otherwise the length of the diagonal PR. (Ans. Area = 8,804.43 m^2, PR = 57.69m)

Q. 10 Write short notes on any four of the following:

(i) Plane and geodetic survey
(ii) Horizontal and level surface
(iii) Variation in magnetic declination
(iv) Fly and reciprocal levelling
(v) Closed and open traverse
(vi) Temporary adjustment of Dumpy level

(Ans. (i) See Sec. 1.4, (ii) See Sec. 5.2, (iii) See Sec. 3.2, (iv) See Sec. 5.6 and 5.10, (v) See Sec. 3.4, and (vi) See Sec. 5.5)

FIELD SURVEYING I, 1990

Group A

Q. 1 (a) What is the type of area best suited to chain surveying? Give reasons. (Ans. See Sec. 2.1)

(b) What are the instruments used in chain surveying? (Ans. See Sec. 2.6)

(c) What do you understand by the term well-conditioned triangles and why are they used? (Ans. See Sec. 2.2)

(d) What is reconnaissance? State its importance in chain surveying. (Ans. See Sec. 2.3)

Q. 2 (a) What are offsets? How are they taken and recorded? Why is it desirable that offsets be as short as possible? (Ans. See Sec. 2.4)

(b) Find the maximum length of an offset for the displacement of a point on a map not to exceed 0.025 cm, given that the offset was laid out 3° from its true direction and the scale of map was 1 cm to 10 m. (Ans. 4.7768 m)

Q. 3 (a) Describe a field book used in chain surveying. (Ans. See Sec. 2.7)

(b) Rule out a page of your answer book in the form of a field book and enter in it the following field readings as obtained while chaining from station C to D.

(i) Length of CD is 35 m.
(ii) The offset length to the corner of a building S at the left of CD is 5 m from chainage 3 m.
(iii) The perpendicular offset of a tree to the right of CD is 6 m from chainage 8 m.
(iv) The perpendicular offset to the corner of a rectangular pond P at the right of CD is 6 m from chainage 10 m.
(v) The perpendicular offset of building S is 7 m from chainage 15 m.
(vi) The perpendicular offset of another corner of pond P is 3 m from chainage 25 m. (Ans. See Sec. 2.8)

Q. 4 (a) Give a list of corrections to be applied to a measurement made with a steel tape. (Ans. See Sec. 1.21)

(b) A steel tape was exactly 30 m long at 18°C when supported throughout its length under a pull of 80 N. A line was measured with a tape under a pull of 120 N, and found to be 801 m long. The mean temperature during the measurement was 26°C. Assuming the tape to be supported at every 30 m compute the true length of the line. Given that cross-sectional area of tape = 0.04 cm^2, mass density of steel = 0.0077 kg/cm^3, coefficient of expansion = 0.0000117 per °C, and modulus of elasticity = 21×10^6 N/cm^2. (Ans. 800.922 m)

Q. 5 Write brief notes with neat sketches where necessary on any four of the following:

(i) Optical square	(See Sec. 2.12)
(ii) Engineer's chain	(See sec. 1.7)
(iii) Closing error	(See Sec. 3.17)
(iv) Reciprocal levelling	(See Sec. 5.10)
(v) Folding-type-4 m levelling staff.	(See Sec. 5.4)
(vi) Clinometer	(See Sec. 1.15)

Group B

Q. 6 (a) Define the following terms:

(i) True meridian
(ii) Magnetic meridian
(iii) True bearing
(iv) Magnetic bearing
(v) Fore and back bearing (Ans. See Sec 3.2)

(b) The following bearings were taken in running a closed compass traverse in an area where local attraction was suspected.

Line	FB	BB
AB	48°25′	230°00′
BC	177°45′	356°00′
CD	104°15′	284°55′
DE	165°15′	345°15′
EA	259°30′	79°00′

(i) State which stations were affected by local attraction and by how much.
(ii) Determine the corrected bearings.

(Ans. (i) + 30′ at A, – 1°5′ at B, + 40′ at C, and (ii) 48°55′, 176°40′, 104°55′, 165°15′, 259°30′)

Q. 7 (a) The bearings of the sides of a traverse ABCDEA are as follows:

Line	FB	BB
AB	105°10′	285°10′
BC	20°20′	200°20′
CD	275°35′	95°35′
DE	179°45′	359°45′
EA	120°50′	300°50′

Compute the interior angles of the traverse and exercise geometric check. (Ans. ∠A = 164°20′, ∠B = 95°10′, ∠C = 75°15′, ∠D = 84°10′, ∠E = 121°5′)

(b) The declination at a place 50 years ago was 3°45′ W. The present declination is 2°15′ E.

If the old magnetic bearing of a line in such an area was recorded as 143°30′, find its present magnetic bearing. (Ans. 137°30′)

(c) State four precautionary measures to be taken while executing compass survey in the field. (Ans. See Sec. 3.19)

Q. 8 (a) Define the following:
Fore sight, backsight, Intermediate sight and benchmark.

(b) Describe the temporary adjustment of a level. (Ans. See Secs. 5.2 and 5.5)

Q. 9 Differentiate between the following with neat sketches where necessary:

(a) GTS and temporary benchmark
(b) Line of collimation and height of instrument

(c) Horizontal and level surface
(d) Closed and open traverse
(e) Check and tie line (Ans. See Secs 5.2, 3.4, and 2.4)

Q. 10 The Following consecutive readings were taken with a dumpy level along a chain line at common intervals of 30 m. The first reading was at a chainage 'O' m where the RL is known to be 150.250.

3.865, 3.345, 2.930, 1.950, 0.855, 3.795, 2.640, 1.540, 1.935, 0.865, 0.665

The instrument was shifted after the sixth and eighth readings. Calculate the RLs of different points and find the difference of level between the first point and the last.

(Ans. 150.770, 151.185, 152.165, 153.260, 150.320, 151.420, 152.49, 152.690, Rise—2.440)

FIELD SURVEYING I, 1989

Group A

Q. 1 Classify surveying on the basis of instruments used and name all the equipment necessary for the field work associated with any one ot them. (Ans. See Secs 1.4 and 2.6)

A plan represents an area of 93,750 m^2 and measures 6 cm × 6.25 cm. Find the scale of the plot and indicate through a sketch how a suitable scale can be constructed to read up to 1.0 m in the plan. (Ans. See Sec. 1.26)

Q. 2 What is a well-conditioned triangle? Explain clearly why they are preferred to ill-conditioned triangles. (Ans. See Sec. 2.2)

The sides of a triangle are 156, 103 and 257 m.
Examine whether the triangle is well-conditioned. (Ans. See Sec. 1.25)

Explain with neat sketches the principle and uses of an optical square. (Ans. See Sec. 2.12)

Q. 3 Describe with sketches how an obstacle which obstructs chaining but not ranging can be overcome in chain survey. (Ans. See Sec. 1.16)

A survey line CD intersects a building. To overcome the obstacle, a perpendicular CE, 87 m long, is set out at D. From E two lines EF and FG are set out at angles of 50° and 65° respectively with ED. Find the lengths EF and EG so that the points F and G are on the prolongation of CD. Also find the obstructed distance DF.

(Ans. EF = 135.349 m, EG = 205.868 m, DF = 103.683 m)

Q. 4 A steel tape was exactly 30 m long at 20°C when supported throughout its length under a pull of 10 kg. A line was measured with tape under a pull of 15 kg and at a mean temperature of 32°C, and found to be 780 m long. Cross-sectional area of tape = 0.03 cm^2, total weight of tape = 0.693 kg α for steel = 11 × 10^{-6} per °C and E for steel = 2.1 × 10^6 kg/cm^2. Compute the true length of the line if the tape was supported during measurement (i) at every 30 m, and (ii) at every 15 m.

(Ans. (i) 780.094 m, and (ii) 780.147 m)

Group B

Q. 5 Describe with the help of a sketch the construction of a dumpy level. Explain the temporary adjustment of level in a field. (Ans. See Sec. 5.4 and 5.5)

Q. 6 The following consecutive readings were taken with a level at intervals of 15 m. The chainage and RL of the first point are 180 m and 18.315 m respectively.

0.915, 1.255, 1.725, 3.055, 1.025, 2.625, 2.935, 3.155, 0.575, 1.505, 2.165

The instrument was shifted after the fourth and eighth readings. Find the RLs of all the points and draw the longitudinal section.

(Ans. 17.975, 17.505, 16.175, 14.575, 14.265, 14.045, 13.115, 12.455)

Q. 7 Describe, along with an example, the difference between the magnetic bearing, true bearing and arbitrary bearing. (Ans. See Sec. 3.2)

The following bearings were observed in traversing with a compass, an area where local attraction was suspected. Find the corrected bearings and included angles of the traverse. Draw the sketch also when AB = 100 m, BC = 100 m and CD = 50 m.

Line	FB	BB
AB	68°15′	248°15′
BC	148°45′	326°15′
CD	224°30′	46°00′
DE	217°15′	38°15′
EA	327°45′	147°45′

(Ans. + 2°30′ at C, + 1° at D;
Corrected bearings = 68°15′, 148°45′, 227°00′, 218°15′, 327°45′)

Q. 8 Write short notes on the following:
(a) Whole circle and reduced bearing
(b) Cumulative and compensating error
(c) Plane and geodetic survey
(d) Closed and open traverse (Ans. See Secs 3.2, 1.19, 1.4 and 3.4)

FIELD SURVEYING I, 1988

Q. 1 The following slope distances were measured along a chain line with a 20 m steel tape:

Slope distance—17.5–19.3–17.8–13.6–12.9
Difference of level
between two ends—2.35–4.20–2.95–1.65–3.25

It was noted afterwards that the tape was 2.5 cm too long. Find the true total horizontal distance. (Ans. 79.815 m)

Q. 2 Describe the different type of chains used in survey, indicating the relative advantages of each.

Explain how the distance between two stations on a more or less plane field can be measured. (Ans. See Secs 1.7 and 1.14)

Q. 3 A 1.2 km long road strip is indicated in a map by a length of 30 cm. Find the scale of plot and indicate through a sketch how a suitable scale can be constructed to read up to 1.0 m in the map. (Ans. See Sec. 1.26)

Q. 4 What is the object of providing a tie line in a chain survey? Describe with neat sketches how an obstacle which obstructs chaining but not ranging can be overcome in chain survey. (Ans. See Secs 2.4, and 1.16)

Q. 5 P and Q are two points 367 m apart on the same bank of a river. The bearings of a tree on the other bank observed from P and Q are N 36°25′ E and N 40°35′W respectively. Find the width of the river if the bearing of PQ is S 86°35′E. (Ans. 227.229 m)

Q. 6 A 20 m steel tape was standardised on flat at a temperature of 20°C and under a pull of 15 kg. The tape was used in catenary at a temperature of 30°C and under a pull of P kg. Cross-sectional area of tape = 0.02 cm^2, total weight of tap = 400g, $E = 2.1 \times 10^6$ kg/cm^2, and $\alpha = 11 \times 10^{-6}$ per °C Find the value of correct distance, if $P = 10$ kg. (Ans. 19.9985 m)

Q. 7 Write short notes on:
(a) Reconnaissance
(b) Sources of error in chaining
(c) Normal tension
(d) Maximum length of offset
(Ans. See Secs 2.3, 1.19, 1.21, and 2.4)

Q. 8 The following consecutive readings were taken with a dumpy level along a chain line at intervals of 15 m. The chainage of the first reading is 165 m and the RL of the last reading 8.085 m. The instrument was shifted after the fourth and ninth readings:

3.150, 2.245, 1.125, 0.860, 3.125, 2.760, 1.835, 1.470, 1.965, 1.225, 2.390, and 3.035 m.

Calculate the RLs of all the points, and also the gradient of the line joining the first and the last points.
(Ans. 6.445, 7.350, 8.470, 8.735, 9.100, 10.025, 10.390, 9.895, 8.730, Rising gradient = 1/82.32)

Q. 9 Write short notes on:
(a) HI method and rise and fall method
(b) Local attraction
(c) Closing error
(d) Fly levelling and Reciprocal levelling
(Ans. See Secs 5.12, 3.2, 3.17, and 5.6 and 5.10)

FIELD SURVEYING II, 1991

1 Write short notes with sketches where necessary on any four of the following:
(a) Optical square
(b) Bowditch rule
(c) Local attraction—detection and elimination
(d) Mass haul curve
(e) Equation of a transition curve
(f) Profile levelling
(Ans. See Secs 2.12, 3.17, 2.2, 12.2, 10.10 and 5.2)

2 (a) Explain the principle of chain surveying. In what conditions is chain surveying more suitable? (Ans. See Sec. 2.1)
(b) A and B are two points 200 m apart on the right bank of a river flowing from east to west. A tree on the left bank is observed from A and B. The bearings of the tree are 20° and 330° respectively as observed clockwise with respect to the north. Find the width of the river. (Ans. 212.4876 m)

3 (a) Differentiate between:
(i) True and magnetic meridian
(ii) Declination and dip (Ans. See Sec. 3.2)

(b) A closed compass traverse ABCD was conducted round a lake and the following bearings were obtained. Determine which of the stations are suffering from local attraction and give the values of the corrected bearings.

Line	FB	BB
AB	74°20′	256°0′
BC	107°20′	286°20′
CD	224°50′	44°50′
DA	306°40′	126°0′

(Ans. + 40′ at A, – 1° at B; Correct FBs = 75°0′, 106°20′, 224°50′, and 306°40′)

4 (a) List out carefully and systematically the field precautions a surveyor should take to ensure good results from levelling field work planned for engineering purposes. (Ans. See Sec. 5.4)
(b) The following is the page of a level book entered in pencil. Some of the entries got erased, and have been marked with crosses. Calculate the missing readings.

Station	BS	IS	FS	Rise	Fall	RL
1	×					150.000
2		2.457		0.827		×
3		2.400				×
4	2.697		×		×	148.070

Station	BS	IS	FS	Rise	Fall	RL
5	×		2.051			148.716
6		2.500				149.784
7		2.896				149.388
8		×			0.124	×
9			2.672			149.612

(Ans. BS : 1.630, 3.568; IS : 3.020; FS : 3.560; Fall : 1.160; RL : 149.173, 149.230, 149.264)

5 (a) What is a contour line? What is the importance of contour maps in civil engineering works?

(b) What considerations would you have while selecting a contour interval?

(c) Discuss the characteristics of a contour. Draw sin table sketches. (Ans. For all three parts, See Secs 6.1 and 6.4)

6 (a) Discuss the advantages and disadvantages of plane table surveying in relation to other methods. (Ans. See Sec. 4.9)

(b) Explain with sketches the following methods of locating a point by plane table survey. Also discuss the relative merits and possible applications of the following methods:

(i) Radiation
(ii) Intersection
(iii) Resection

(Ans. See Sec. 4.5 for all three parts)

7 (a) Explain the method of computation of volume by the:

(i) Trapezoidal rule
(ii) Prismoidal rule (Ans. See Sec. 8.3 for both parts)

(b) Calculate the volume of embankment, given the following cross-sectional areas at 20 m intervals:

Distance (m)—0–20–40–60–80–100–120
Area (m^2)—10–40–64–72–160–180–260

Use: (i) The trapezoidal formula
(ii) The prismoidal formula

(Ans. (i) 13,020 m^3; (ii) 12,573.33 m^3)

8 (a) Explain with sketches:

(i) A simple curve
(ii) A compound curve
(iii) A reverse curve. (Ans. See Sec. 10.3)

(b) Two straight lines intersect at chainage 1,150.50 m, the angle of intersection being 60°. If the radius of the curve is 500 m, determine:

(i) Tangent length
(ii) Length of the curve
(iii) Chainages of points of curvature and tangency
(iv) Length of long chord
(v) Degree of curve

(Ans. (i) 866.0254 m, (ii) 1,047.19 m, (iii) 284.47 m, 1331.66 m, (iv) 866.025 m, and (iv) 3°26′18″)

9 (a) Discuss different methods of expressing gradient. What is rate of change of gradient? (Ans. See Sec. 10.10)

(b) What are the permanent adjustments of a dumpy level? Explain how they are done in the field. (Ans. See Sec. 5.20)

10 (a) A man travels due west from a point A and reaches point B. The distance between A and B is 139.6 m. Calculate the latitude and departure of the line AB.

(Ans. Latitude = 0, and departure = – 139.6 m)

(b) What is closing error in a theodolite traverse? How would you distribute the closing error? (Ans. See Secs 9.17 and 9.19)

(c) In a closed traverse, the latitudes and departures of the sides were calculated, and it was observed that:

Latitude = 1.39 m
Departure = – 2.17 m

Calculate the length and bearing of closing error.

(Ans. Length = 2.58 m, and bearing = N 57°21′29″W)

FIELD SURVEYING II, 1990

Group A

1 (a) State the possible sources of error in levelling. (Ans. See Sec. 5.16)

(b) The following consecutive readings were taken with a dumpy level:

1.895, 1.500, 1.865, 2.570, 2.990, 2.020, 2.410, 2.520, 2.960 and 3.115

The level was shifted after the fourth, sixth and ninth readings. The reduced level at the first point was 30.500 m. Rule out a page of your answer book as a level book and fill up all columns. Use the collimation system and apply the usual arithmatical checks.

(Ans. 30.895, 30.530, 29.825, 30.795, 30.685, 30.245)

2 (a) Describe with sketches how the collimation adjustment of a dumpy level is made using the two-peg method. (Ans. See Sec. 5.20)

(b) A level was set up midway between two pegs A and B, 100 m apart and the staff readings on pegs A and B were 2.325 and 3.020 respectively. The level was then set up near peg A and the staff readings on pegs A and B were 1.520 and 2.285 respectively. Find out the error in the collimation line and the correct reading on peg B from the second position of the instrument.

(Ans. Collimation error = + 0.070 m, correct staff reading at B = 2.215 m)

3 (a) Name the different methods of plane table surveying.

(b) Describe the method of intersection.

(c) What is a 'three-point problem'?

(d) Describe the solution of a three-point problem by plane table.
(Ans. See Sec. 4.5 and 4.6)

4 (a) Explain the terms:
(i) Contour gradient
(ii) Interpolation of contours
(b) What are different methods of locating contours? Describe their merits and demerits.
(c) What are the uses of a contour map?
(Ans. See Secs 6.1, 6.3 and 6.5)

5 (a) Mention how you would set up a theodolite and measure horizontal and vertical angles.
(b) What would you mean by face left and face right observations? How can you change the face? Explain the errors eliminated by change of face.
(c) Give five possible errors in conducting a traverse with a vernier theodolite. (Ans. See Secs 9.6, 9.2 and 9.16)

6 Write short notes on any four of the following with sketches where necessary:
(a) GTS and temporary benchmark
(b) Limitations of Simpson's rule
(c) Mass diagram
(d) Tacheometer
(e) Transition curve
(f) Balancing of traverse
(Ans. See Secs 5.2, 7.8, 12.2, 11.1, 10.10 and 9.19)

Group B

7 (a) What is traverse surveying? How does it differ from chain surveying? Distinguish between a closed and an open traverse.
(Ans. See Secs 3.4, 3.5)
(b) The lengths and bearings of a traverse ABCD are observed as follows:

Line	*length (m)*	*WCB*
AB	470	336°20′
BC	1780	15°20′
CD	1080	140°36′

Compute the length and bearing of line AD.
(Ans. 1,630.61 m, N 36°23′43″ W)

8 (a) What are the elements of a simple circular curve? Explain in detail how a simple curve is designated. (Ans. See Secs 10.4 and 10.5)
(b) Two straight lines AB and BC intersect at chainage 950.0 m, the intersection angle being 140°. It is desired to connect these two straights by a simple circular curve of 5°. Calculate the radius of the curve and the chainage of the tangent points if the length of the unit chord is 30 m.
(Ans. R = 343.8 m, ch of T_1 = 824.87 m, ch of T_2 = 1,064.87 m)

9 A series of offsets were taken from a chain line to an irregular boundary at intervals of 30 m in the following order:

0, 7.4, 5.6, 6.3, 6.9, 7.5, 8.3, 0 m

Compute the area between the chain line, the boundary line and the end offsets by:
(a) The trapezoidal rule
(b) Simpson's rule
(c) The average ordinate rule
(Ans. (a) 1,260 m^2, (b) 1,181.0 m^2, and (c) 1,102.5 m^2)

10 (a) The cross-section of a stream 30 m wide is measured by means of soundings taken 5 m apart, the depths recorded being 0, 1.5, 2.0, 3.5, 2.3, 1.0, and 0 m. The mean velocity is observed to be 3.4 m/s. Compute the discharge of the stream. (Ans. 175.10 m^3/s)
(b) Calculate using the prismoidal formula the capacity of a reservoir between contours 100 m and 112 m from the following data:

Contour (m)	Area enclosed by contour (m^2)
100	100,000
103	115,000
106	118,000
109	124,000
112	130,000

(Ans. 1,416,000 m^3)

11 (a) Draw a neat sketch of a transit theodolite and name its different parts.
(b) What are the temporary adjustments required for it?
(Ans. See Secs 9.8 and 9.6)

FIELD SURVEYING II, 1989

Group A

1 The following is the page of a level field book.
Fill in the missing readings and calculate the reduced levels of all points.
Apply the usual checks.

Station	BS	IS	FS	Rise	Fall	RL	Remarks
1	3.250					249.260	BM
2	1.755		×		0.750		CP
3		1.950					
4	×		1.920				CP
5		2.340		1.500			
6		×		1.000			
7	1.850		2.185				CP
8		1.575					
9		×					
10	×		1.895		1.650		CP
11			1.350	0.750			Last point

(Ans. BS : 3.840, 2.100; IS : 1.340, 0.245; FS : 4.000; RL : of last point: 250.705)

2 (a) What are the two permanent adjustments of a dumpy level? What are the objects of such permanent adjustments? (Ans. See Sec. 5.20)
(b) The following observations were made during the testing of a dumpy level:

Instrument at	Staff reading at	
	A	B
A	1.725	2.245
B	2.145	3.045

Distance between A and B = 200 m
(a) Is the instrument in adjustment?
(b) To what reading should the line of collimation be adjusted when the instrument is at B?
(c) If the RL of A = 450.00 m, what should be the RL of B?
(Ans. (a) Collimation line not in adjustment
(b) Collimation error = – 0.190 m
(c) RL of B = 449.290 m)

3 (a) What is plane table surveying?
(b) When would you recommend it?
(c) What are the accessories required in a plane table survey?
(d) What do you understand by the term "orientation of a plane table"?
(e) Describe with a neat sketch a suitable method of orientating a plane table.
(f) State any two disadvantages of plane table surveying.
(Ans. See. Secs 4.1, 4.2, 4.3 and 4.9)

4. (a) What are the different methods used for computing the cubic contents of cutting and embankment? (Ans. See Sec. 8.3)
(b) A railway embankment is 10 m wide and has side slopes in the ratio 2 : 1. Assuming the ground to be level in a direction transverse to the centre line, calculate the volume contained in a length of 150 m by the prismoidal formula, the central heights at 30 m intervals being 2.4, 3.0, 3.5, 4.0, 3.75 and 2.75 m respectively.
(Ans. 8,625.20 m^3)

5. (a) What is a 'Three-point problem'?
(b) Describe with a neat sketch Bessel's graphical method of solving a three-point problem. (Ans. See Sec. 4.6)

6. Write short notes, with neat sketches where necessary, on any four of the following:
(a) Contour and contour interval
(b) Profile levelling
(c) Latitude and departure
(d) Lift and lead
(e) Vertical curve
(f) Superelevation
(g) GTS benchmark
(Ans. See Secs 6.1, 5.6, 9.18, 8.6, 10.11, 10.2 and 5.2)

Group B

7 Differentiate, with neat sketches where necessary, between the following:
(a) Compound and reverse curves
(b) Open and closed traverse
(c) Whole circle and reduced bearings
(d) Radiation and intersection methods.
(Ans. See Secs 10.3, 3.5, 3.2 and 4.5)

8 (a) What are the temporary adjustments of a theodolite?
(b) What would you mean by 'face left' and 'face right' observations?
(c) While undertaking a setting out operation with a theodolite, a line BC is to be set out at a station point B in such a manner that BC deviates to the right of another line AB by an angle 75°30′. Stations A and B are already marked on the field. Describe briefly how you would set out the line BC in the field.
(Ans. See Secs 9.6, 9.2 and 9.10)

9 (a) What is reciprocal levelling? Explain with neat sketches.
(b) What are the two advantages of reciprocal levelling?
(c) Show with neat sketches the characteristic features of contour lines for any three of the following:
(i) a pond, (ii) a hill, (iii) a ridge, (iv) a valley, and (v) a vertical cliff.
(Ans. See Secs 5.10 and 6.4)

10 Two tangents intersect at a chainage at 1,190 m, the deflection angle being 36°. Calculate all the data necessary for setting out a curve of radius 300 m by the deflection angle method using a theodolite having a least count of 20″. The peg interval is 30 m.
(Ans. TL = 97.47 m, (T.L. = Tangent length)
CL = 188.52 m, (C.L. = Curve length)
chainage of T_1 = 1,092.53,
chainage of T_2 = 1,281.05 m,
δ for FC = 2° 51′53″, δ = Deflection angle
Follow the detailed procedure shown in Sec. 10.7)

11 A line AC of length 2 km was required to be set out at right angles to a given line AB. This was done from A to C as follows:

Line	*Length*	*Bearing*
AB	—	360°
AD	760 m	120°
DE	500 m	90°
EF	600 m	105°

Compute the length and bearing of FC. Draw the required diagram.
(Ans. Bearing of FC = N 26°6′17″ E, length of FC = 596°09 m)

12. (a) What is a transition curve?
(b) Where is a transition curve provided and why?
(c) Calculate the length of the vertical curve connecting two uniform

grades of + 0.5% and – 0.7%, given a rate of change of grade of 0.1% per 30 m.

Calculate the reduced level of the vertex of the vertical curve, given that the reduced level of the point of intersection is 350.75 m.

(Ans. See Sec. 10.10 for parts (a) and (b).
(c) Length of vertical curve = 360 m,
Tangent correction at centre = 0.54 m,
RL of vertex = 350.21 m)

FIELD SURVEYING II, 1988

Group A

1. In running fly levels from a benchmark of RL 140.602, the following readings were obtained:

   BS—1.543, 2.694, 1.416, 2.923
   FS—0.574, 1.236, 0.596

   From the last position of the instrument, six pegs at 20 m intervals are to be set out on a uniform rising gradient of 1 in 50; the first peg is to have an RL of 144.000.

   Calculate the staff readings on the pegs, as also the RLs of the pegs.
   (Ans. 2.772, 2.372, 1.972, 1.572, 1.172, 0.772, 144.000, 144.400, 144.800, 145.200, 145.600, 146.000)

2. (a) What are the effects of the earth's curvature and atmospheric refraction on levelling?
   (b) A level is set up at P on a line AB, 50 m from A and 1,000 m from B. The backsight on A is 0.694 m and the foresight on B 3.018 m. Find the true difference in level between A and B.
   (c) What are the possible sources of error in levelling?
   (Ans. (a) See Sec. 5.8,
   (b) 2.2567 m (fall from A to B), and
   (c) See Sec. 5.16)

3. (a) Explain, with neat sketches, the intersection method of plane tabling for locating details.
   (b) Discuss the advantages and disadvantages of plane table surveying in comparison with other methods. (Ans. See Secs 4.5 and 4.9)

4. (a) (i) Define contour.
   (ii) What is a contour interval?
   (iii) State any four uses of contour maps.
   (b) The following offsets were taken from a chain line to a hedge:

   Distance (m)—0–8–16–24–32–40–48–56–64
   Offset (m)—3.76–4.32–5.44–4.88–3.84–3.36–3.00–2.52–1.84

   Compute the area in square metres included between the chain line, the hedge and the end offsets by using Simpson's rule.
   (Ans. (a) See Art 6.1 and 6.3; (b) 241.28 m^2)

5 (a) State the two-point problem.
(b) Explain, with neat sketches, the procedure for solving a two-point problem in plane table survey. (Ans. See Sec. 4.6)

6 Write short notes, with sketches where necessary, on any four of the following:
(a) Reciprocal levelling
(b) Height of instrument
(c) Contour gradient
(d) Check line
(e) Transition curve
(f) Tacheometer
(g) Mass diagram
(Ans. See Secs 5.10, 5.2, 6.7, 2.4, 10.10, 11.1 and 12.2)

Group B

7 (a) Describe how you would set up a theodolite at a given station.
(b) Describe, step by step, how you would measure a horizontal angle by a theodolite by the repetition method.
(c) State what errors will be eliminated by this method.
(Ans. See Secs 9.6 and 9.8)

8 Differentiate, with neat sketches where necessary, between the following:
(a) Consecutive and independent coordinate
(b) Trapezoidal and Simpson's rules
(c) GTS and temporary benchmarks
(d) Simple and compound curves
(Ans. See Secs. 9.18, 7.7, 7.8, 5.2, and 10.3)

9 ABCD is a closed traverse in which the bearing of AD has not been observed and the length of BC has been forgotten to be recorded. The rest of the field records are as follows:

Line	*Bearing*	*Length (m)*
AB	181°.18′	335
BC	90°.00′	—
CD	357°36′	408
DA	—	828

Calculate the bearing of AD and the length of BC. Draw the traverse.
(Ans. Length of BC = 849.48 m,
Bearing of AD = N 84°57′45″ E)

10 (a) Name the different methods for calculation of the area of a surface of land.
(b) An excavation is to be made for a reservoir 30 m long, 20 m wide at the bottom, and 3.5 m deep. The sides of the excavation slope are 2 horizontal to 1 vertical. Assuming the surface of the ground to be level before excavation, calculate the volume of excavation by using the prizmoidal formula.

(c) For what purposes is a mass diagram generally used?

(Ans. (a) See Secs 7.2, 7.4, 7.5, 7.6, 7.7 and 7.8, (b) 3,553.66 m^3, and (c) See Sec. 12.2)

11 (a) What is vertical curve and why is it provided?

(b) Describe, step by step, the method of setting out a circular curve by perpendicular offsets from the tangent with help of only chain and tape. (Ans. See Secs 10.11 and 10.6.)

12 Two straights meet at an apex angle 126°48′ and are to be joined by a circular curve of raidus 300 m. Calculate the data necessary to set out the curve by a theodolite having a least count of 20″ using a 30 m chord. The chainage at the point of intersection is 510.23 m.

(Ans. Tangent length = 150.23 m, curve length = 278.55 m, chainage of T_1 = 360.00 m, chainage of T_2 = 638.55 m.

Follow the detailed procedure shown in Sec. 10.7)

Appendix B

Diploma in Engineering Examinations-Western Region

SURVEYING, NOVEMBER 1990

Q. 1 Answer any ten of the following:

(a) State the purpose of the prism provided in a prismatic compass

(Ans. The prism is provided in a prismatic compass for taking the readings on the graduated ring, where the figures are written inverted)

(b) State the use of the change point in levelling work (Ans. Sec Sec. 5.2)

(c) State the advantages and disadvantages of providing an anallatic lens. (Ans. See Sec. 11.4)

(d) Define latitude and departure (Ans. See Sec. 9.18)

(e) Define axis of bubble tube and benchmark. (Ans. See Sec. 5.2)

(f) Draw the conventional signs for the following:

(i) Barren land (ii) An Orchard (Ans. See Sec. 2.10)

(g) Draw the contours of the following:

(i) A hill (ii) A vertical cliff (Ans. See Sec. 6.4)

(h) The value of the smallest division on a main scale is 1/2 degree and the number of divisions on the vernier scale is 60. Find the least count of the vernier. (Ans. 30″)

(i) State the purpose of the location sketch (reference sketch) of a survey station (Ans. See Sec. 2.9)

(j) State the methods of plane table survey. (Ans. See Sec. 4.5)

(k) List out the various instruments required for measuring distances.

(Ans. (i) 20 m or 30 m chain—1,
(ii) Arrows—10,
(iii) Metallic tape (15 m)—1,
(iv) Ranging rods—3,
(v) Optical square—1, and
(vi) Line ranger—1)

(l) State the use of the line ranger.
(Ans. The line ranger is used to fix intermediate points on a chain line by a surveyor all by himself. It is a small reflecting instrument consisting of two right-angled prisms placed one above the other. When a point is just on the chain line, the surveyor will find the images of the ranging rods at the end stations to coincide exactly)

Q. 2 Solve any two of the following:

(a) (i) A 20 m chain was found to be 14 cm too long after chaining 1,500 m. The same chain was found to be 25.5 cm too long after chaining a total distance of 3,200 m. Find the correct value of the total distance chained assuming the chain was accurate at the commencement of chaining. (Ans. 3,222.04 m)

(ii) State the use of alidade in plane table survey. (Ans. See Sec. 4.2)

(b) The following table gives the FB, and BB of the sides of a closed compass survey:

Line	*FB*	*BB*
PQ	45°00′	226°15′
QR	130°30′	310°00′
RS	184°30′	4°30′
SP	290°00′	109°15′

Find: (i) The stations affected by location attraction,
(ii) The corrected bearings of the lines, and
(iii) The interior angle of traverse,
(iv) Draw the traverse.

(Ans. (i) + 45′ at P and − 30′ at Q,
(ii) FB of PQ = 45°45′,
FB of QR = 130°00′,
FB of RS = 184°30′,
FB of SP = 290°.00′, and
(iii) $\angle P$ = 64°15′, $\angle Q$ = 95°45′,
$\angle R$ = 125°30′, and $\angle S$ = 74°30′)

(c) (i) Convert the following WCBs into QBs and QBs to WCBs.
179°40′, 265°30′, N 10°20′W, S 70°30′E
(Ans. S0°20′E, S85°30′W, 349°40′, 109°30′)

(ii) What do you understand by the closing error of a compass traverse? Show how it can be adjusted by the graphical method. (Ans. See Sec. 3.17)

Q. 3 Solve any two of the following:

(a) (i) State the temporary adjustments to be made in a transit theodolite. (Ans. See Sec. 9.6)

(ii) Describe the method of prolonging a line with a transit theodolite. (Ans. See Sec. 9.13)

(b) The length and bearings of a traverse LMNO are observed as follows:

Line	*Length (m)*	*WCB*
LM	470	336°20′
MN	1,780	15°20′
NO	1,080	140°36′

Compute the length and bearing of line OL.
(Ans. 1,630°62 m, S 36°23′ W)

(c) Describe the permanent adjustment of a transit theodolite. (Ans. See Sec. 9.23)

Q. 4 Solve any two of the following:

(a) A page of an old level book was required to be consulted but found to be damaged. Find out the missing readings marked with a cross and complete the level book page. Apply the usual arithmetical checks.

Station	BS	IS	FS	HI	RL	Remarks
1	3.400			×	×	BM
2		×			192.000	
3	3.900		2.550	×	×	CP
4		3.400			191.300	
5		×			197.000	Staff inverted
6			×		192.300	Last point

(Ans. IS = 1.350, −2.300
(A negative sign indicates an inverted staff).
FS = 2.400
HI = 193.350, 194.700
RL = 189.950, 190.800)

(b) Describe the construction of a dumpy level along with a sketch, and name the different components of the instrument. (Ans. See Sec. 5.4)

(c) (i) Describe the process of profile levelling. (Ans. See Sec. 5.6)

(ii) Give four uses of a contour map. (Ans. See Sec. 6.3)

Q. 5 Solve the following:

(a) State the advantages and disadvantages of plane table surveying. (Ans. See Sec. 4.9)

(b) Two straight lines AB and BC intersect at chainage 2,303 m, the deflection angle being 12°. Calculate all the necessary data for setting out a circular curve of radius 400 m, by the deflection angle method, taking a peg interval of 30 m.

(Ans. Tangent length = 42.04 m,
curve length = 83.78 m,
chainage of 1st TP = 2260.96 m, and
chainage of 2nd TP = 2,344.74 m
Follow the detailed proceduce shown in Sec. 10.7)

Q. 6 A tacheometer was set up at a station A and the following readings were obtained on a vertically held staff:

Station	Staff station	Vertical angle	Hair readings
A	B.M.	– 6°30′	0.900, 1.160, 1.420
A	B	– 2°20′	1.140, 1.230, 1.320

The constants of the instrument were 100 and 0.1. Find the horizontal distance AB and RL of B if the RL of BM is 200.00 m.

(Ans. AB = 18.07 m
RL of B = 205.050 m)

SURVEYING, MAY 1990

Q. 1 Answer any ten of the following.
(a) State the principle of surveying. (Ans. See Sec. 1.5)
(b) State the different uses of theodolite. (Ans. See Sec. 9.1)
(c) List out the instruments required for city surveying.
Ans. (i) 20 m or 30 m chain—1,
(ii) Arrows—10,
(iii) 15 m metallic tape—1,
(iv) Ranging rods—3,
(v) Line ranger—1,
(vi) Optical square or cross staff—1,
(vii) Levelling instrument with stand—1,
(viii) Levelling staff (4 m)—1,
(ix) Prismatic compass—1, and
(x) Transit theodolite—1.
(d) Name the methods of plane table surveying. (Ans. See Sec. 4.5)
(e) State the least count of:
(i) Levelling staff, (ii) metallic tape, (iii) steel tape, and (iv) metric chain
(Ans. (i) 5 mm, (ii) 20 cm, (iii) 20 cm, and (iv) 20 cm)
(f) Draw the contours of the following:
(i) hill, and (ii) valley line (Ans. See Sec. 6.4)
(g) Draw the conventional signs for the following:
(i) chain line, and (ii) marshy land. (Ans. See Sec. 2.10)
(h) State the purpose of the lifting lever in the prismatic compass. (Ans. See Sec. 3.7)
(i) State the principle used in the construction of the optical square. (Ans. See Sec. 2.12)
(j) Define: (i) Foresight, and (ii) backsight. (Ans. See Sec. 5.2)
(k) Define contour and state the methods of interpolation of contours. (Ans. See Secs 6.1 and 6.6)
(l) Define parallax and state how it is eliminated. (Ans. See Sec. 5.2)

Q. 2 Solve any two of the following:
(a) (i) A line was drawn to a magnetic bearing of 212° on an old map when the magnetic declination was 4° East. To what M. B. should it be set now if the present magnetic declination is 10° West? (Ans. 226°)
(ii) The bearing of a line AB is 40°30′, and the angle ABC is 75°40′. Calculate the bearing of the line BC. (Ans. 296°10′)
(b) (i) Explain the direct method of chaining on sloping ground. (Ans. See Sec. 1.15)
(ii) What are the possible sources of cumulative error in chaining survey lines? (Ans. See Sec. 1.19)
(c) Explain in detail how a chain is tested and adjusted in the field. (Ans. See Sec. 1.10)

Q. 3 Solve any two of the following:
(a) (i) State the error eliminated by the method of repetition. (Ans. See Sec. 9.8)
(ii) Explain how you would measure the bearing of a line with a theodolite. (Ans. See Sec. 9.11)
(b) The following are the latitudes and departures of a closed traverse ABCDE. Determine the independent coordinates of all the points.

Line	*Latitude*	*Departure*
AB	+ 84.59	+ 40.20
BC	+ 68.88	– 80.06
CD	– 23.40	– 76.80
DE	– 67.65	+ 27.67
EA	– 62.42	+ 88.99

(Hint: Consider station A as the origin, and assume its Y-coordinate to be + 100.00 and X-coordinate + 200.00. Then the independent coordinates of stations B, C, D and E may be obtained by calculating the algebraic sum of the given latitudes and departures of lines AB, BC, CD, DE and EA. Finally, the coordinates of station A will work out to be + 100.00 and + 200.00. This is the check.)
(c) (i) What is an anallatic lens and where is it fitted? What are its advantages? (Ans. See Sec. 11.4)
(ii) Define the following terms:
(i) Transiting, (ii) swinging, (iii) face left, observation, and (iv) telescope normal. (Ans. See Sec. 9.2)

Q. 4 Solve any two of the following:
(a) The following consecutive readings were taken with a level and 4 m staff on a continuously sloping ground at common intervals of 30 m:

0.750, 1.450, 2.010, 2.600, 2.975, 0.535, 1.730, 2.950, 3.500, 0.700, 1.860.

The RL of the first point was 100.000 m.

Calculate the RLs of all the points, and the gradient of the line joining the first and last points.

(Ans. Falling gradient = 1/37.79)

(b) (i) Give the characteristics of contours. (Ans. See Sec. 6.4)

(ii) Explain the method of orientation by backsighting.

(Ans. See Sec. 4.3)

(c) (i) Describe briefly how the axis of the bubble tube can be adjusted in the dumpy level. (Ans. See Sec. 5.20)

(ii) What do you understand by route surveying? What is the information to be collected during reconnaissance survey?

(Hints: Survey work along the alignment (centre line or profile line) of any project involving roads, railways, irrigation, etc. is known as route surveying. It involves the preparation of a route survey map showing the natural features for a strip of land on both sides of the alignment. Longitudinal and cross-sectional levelling are done to prepare a long-section and cross-section for computation of the volume of earth work.

During reconnaissance the following points should be noted:

(a) The general features of the area
(b) Crossing points with roads, rivers, canals, etc.
(c) The soil survey report
(d) The water table along the alignment
(e) Availability of construction material and labour
(f) Places of importance to be connected
(g) The recommendation of a tentative alignment

Q. 5 Solve any two of the following:

(a) A road surveyor recorded the following data for a simple circular curve:
Length of long chord = 120 m
Radius of curve = 200 m
Calculate the ordinates at 12 m intervals

(Ans. Follow the example given in Sec. 10.6, sub-section A)

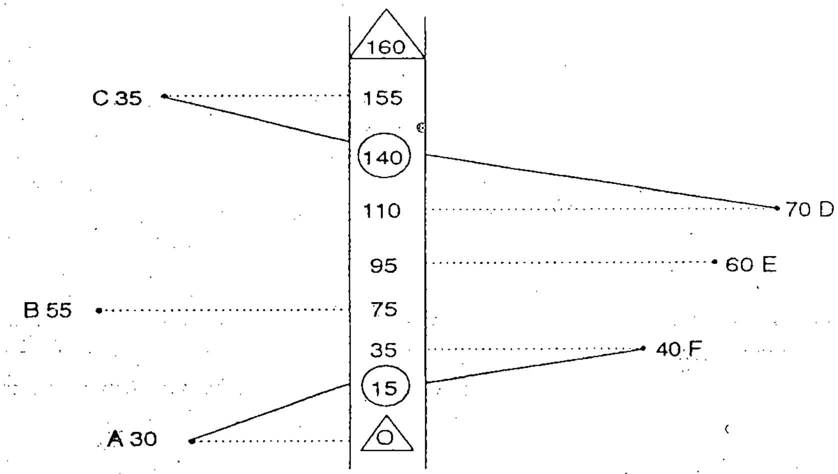
(b) (i) The following readings were obtained when an area was measured by planimeter, the tracing arm being so set that one revolution of the wheel measured 10 cm^2 on paper.

IR = 9.800
FR = 4.390

The position of the anchor point is outside the figure. The zero mark passes across the points once in the clockwise direction. Find the area of the figure. (Ans. 45.9 cm^2)

(ii) Describe the method of determining the multiplying constant of a tacheometer. (Ans. See Sec. 11.3)

(c) Plot the data relevant to the following cross-staff survey of a field ABCDEFA, and calculate the required area.



All measurement in metres

(Ans. Area = 12,687.5 m^2)

Q. 6 Solve any two of the following:

(a) A tacheometer fitted with an anallatic lens was set up at station B and the following observations were recorded. The value of the multiplying constant is 100.

Station	Station sighted	Bearings	Vertical angle	Staff readings
B	A	325°30′	+ 3°20′	0.750, 1.000, 1.250
B	C	55°30′	− 2°30′	1.250, 2.000, 2.750

(i) Find the distance AC

(ii) Find the difference of level between A and C.

(Ans. (i) 157.785 m, and (ii) 10.439 m)

(b) Describe the permanent adjustment of a transit theodolite.

(Ans. See Sec. 9.23)

SURVEYING, NOVEMBER 1989

Q. 1 Answer any ten of the following:

(a) State the basic difference between plane and geodetic survey.

(Ans. See Sec. 1.4)

(b) State the instruments required for plane table survey. (Ans. See Sec. 4.2)

(c) Name the fundamental lines of a theodolite. (Ans. See Sec. 9.23)

(d) Draw the conventional signs for the following:
(i) A railway bridge, and (ii) an embankment (Ans. See Sec. 2.10)

(e) Draw the contour for the following:
(i) A lake, and (ii) a ridge line. (Ans. See Sec. 6.4)

(f) State the purpose of the rider provided in the prismatic compass.
(Ans. A rider made of brass or silver coil is provided to counterbalance the dip of the needle)

(g) State the effect of local attraction on the calculation of included angle from observed bearings.
(Ans. If the bearings of the lines are taken at the same time with the same instrument, then local attraction will not affect correctness of the values of included angles)

(h) Define orientation, and name two methods of orientation. (Ans. See Sec. 4.3)

(i) Define least count and state how the least count of a theodolite is worked out. (Ans. See Sec. 9.2)

(j) State any four uses of a contour plan. (Ans. See Sec. 6.3)

(k) Why should backsight and foresight be equidistant from the instrument? (Ans. See Sec. 5.7)

Q. 2 Solve any two of the following:

(a) The ovserved fore and back bearings of the lines of a closed compass traverse ABCD are:

Line	*FB*	*BB*
AB	104°30′	284°30′
BC	48°00′	226°00′
CD	290°30′	115°15′
DA	180°15′	357°30′

(i) Find the local attraction at all the stations.
(ii) Find the corrected FB.
(iii) Calculate the interior angles.

(Ans. (i) 0° at A, 0° at B, + 2° at C, – 2°– 45′ at D,
(ii) 104°30′, 48°00′, 292°30′, 177°30′, and
(iii) $\angle A = 107°$, $\angle B = 123°–30′$, $\angle C = 64°– 30′$, $\angle D = 65°00′$.)

(b) (i) Draw a neat sketch of an optical square and describe its working. (Ans. See Sec. 2.12)
(ii) Explain, with a neat sketch, the method of reciprocal ranging. (Ans. See Sec. 1.8)

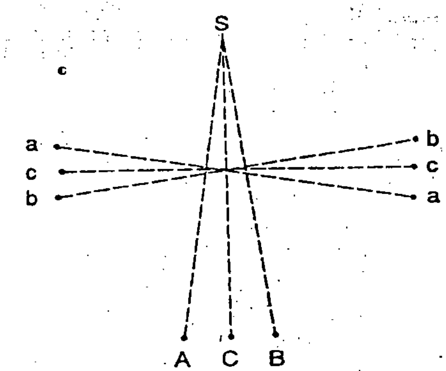
(c) (i) Calculate the interior angle ABC if the bearing of AB = 120°30′ and that of BC = 110°00′. (Ans. 169°30′)
(ii) The true bearing of a line AB is 36°15′. Find its magnetic bearing when the declination is 4°15′ West. (Ans. 40°30′)

(iii) The magnetic bearing of a line CD is 193°30′. Find its true bearing when the magnetic declination is 3°15′ East. (Ans. 196°45′)

Q. 3 Solve any two of the following:

(a) (i) State the steps involved in using a theodolite for measuring deflection angle. (Ans. See Sec. 9.10)
(ii) State the possible sources of errors in theodolite survey. (Ans. See Sec. 9.16)

(b) What is a spire test? Give the purpose, and the required adjustment for it.
(Ans. The test by which the horizontal axis is established to be perpendicular to the vertical axis is known as the spire test. The purpose of this adjustment is to make the vertical axis truly vertical when the instrument is levelled with respect to the plate bubble and the altitude bubble.



Test: (i) The theodolite is set up at some distance from a building. Let us consider a point S on the wall at some considerable height. Both the clamp screws are kept fixed.
(ii) The point S is sighted. Then the telescope is depressed and a point A is marked near the base of the wall.
(iii) By changing the face, S is again bisected. The telescope is depressed, and it may be found that A is not bisected now. Then a new point B is marked.

Adjustment:
(i) Now a point C is marked at midway between A and B.
(ii) C is bisected and the clamp screws are kept fixed.
(iii) The telescope is raised, but it is seen that S is not bisected.
(iv) By raising or lowering the adjustable end of the trunnion axis, the line of sight is made to pass exactly through S.

The adjustment is now complete.

(c) An abstract from a traverse sheet for a closed traverse is given below.

Line	Length (m)	Latitude (m)	Departure (m)
AB	200	– 173.20	+ 100.00
BC	130	0.00	+ 130.00
CD	100	+ 86.60	+ 50.00
DE	250	+ 250.00	+.000
EA	320	– 154.90	– 280.00

(i) Find the corrections to be applied to the latitude and the departure for balance of the survey by Bowditch's rule.

(ii) Find the corrected latitude and departure of each line.

(Ans. Total error in latitude = + 8.50
Total error in departure = 0.00

So, the departures of the lines do not require any adjustment.

Now, Total correction in latitude = – 8.50 (negative)

Correction for latitude

$$AB = -8.50 \times \frac{200}{1,000} = -1.700$$

$$BC = -8.50 \times \frac{130}{1,000} = -1.105$$

$$CD = -8.50 \times \frac{100}{1,000} = -0.850$$

$$DE = -8.50 \times \frac{250}{1,000} = -2.125$$

$$EA = -8.50 \times \frac{320}{1,000} = -2.720$$

Total = – 8.50

Corrected Latitude

AB = – 173.20 – 1.700 = – 174.900
BC = 0.00 – 1.105 = – 1.105
CD = + 86.60 – 0.850 = + 85.750
DE = + 250.00 – 2.125 = + 247.875
EA = – 154.90 – 2.72 = – 157.620

Q. 4 Solve any two:

(a) The following consecutive readings were taken with a dumpy level.

3.705, 3.300, 1.875, 0.790, 3.805, 2.590, 1.485, 2.100, 0.830, 0.635

The level was shifted after the fifth and eighth readings. The first reading was taken on a BM of RL 140.000 m.

(i) Prepare a page of level book and enter all the readings.

(ii) Calculate the RLs of all the points.

(iii) Apply the usual arithmetical check.
(Ans. RLs—140.405, 141.830, 142.915, 139.900, 140.005, 140.390, 140.585)

(b) (i) State the possible sources of error in levelling work.
(Ans. See Sec. 5.16)

(ii) What is fly levelling? When is it carried out.
(Ans. See Sec. 5.6)

(c) Describe in detail the temporary adjustment of a dumpy level.
(Ans. See Sec. 5.5)

Q. 5 Solve any two:

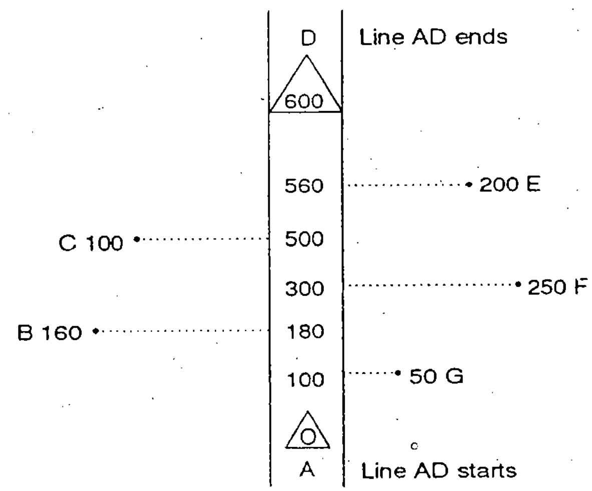
(a) (i) Describe the method of intersection in plane tabling.
(Ans. See Sec. 4.5)

(ii) State the purposes of city surveying.
(Ans. The following are the purposes of city surveying:

(a) For preparing a general map of the city to know the topography,
(b) For laying out plots, streets, lanes, parks, etc.,
(c) For laying out water supply pipelines, sewer lines, telephone lines, electricity lines, etc., and
(d) For monumenting reference points)

(b) Describe the method of determining the constant of a tacheometer.
(Ans. See Sec. 11.3)

(c) Plot the data for the following cross-staff survey of a field and calculate its area, namely the area of ABCDEFGA, in hectares.
(Ans. 15.6 hectares)



Q. 6 Solve any two:

(a) A tacheometer fitted with an anallatic lens and having a multiplying constant of 100 was used and the following observations were made with the staff held vertically.

Inst. Stn	HI	Staff at	Vert. angle	Staff readings
P	1.500	B.M.	– 3°– 10′	1.350, 1.480, 1.610
P	1.500	Q	+ 2°– 45′	1.340, 1.650, 1.960

The RL of the BM is 175.000 m. Calculate:
(i) The RL of P and Q
(ii) The distance PQ

(Ans. (i) RL of P = 175.046 m,
RL of Q = 176.367 m, and
(ii) Distance PQ = 61.857 m)

(b) Two straights AB and BC intersect at chainage 1,000 m, the deflection angle being 30°0′. Calculate all the necessary data for setting out a simple curve with radius 350 m by the deflection angle method, taking a peg interval of 30 m.
(Ans. Follow the procedure shown in Sec. 10.7)

(c) State the advantages and disadvantages of plane table survey.
(Ans. See Sec. 4.9)

SURVEYING, NOVEMBER 1988

Q. 1 Answer any ten of the following:

(a) State the meaning of the term "drawing to scale".
(Ans. The process of preparing such a drawing in which the actual size of an object is increased or decreased by a predetermined ratio, is termed "drawing to scale". The process by which the acutal length of an object is reduced or increased to some ratio on a drawing is termed the "scale" of the drawing.

(b) State the principle of the diagonal scale.
(Ans. The principle of the diagonal scale is based on that of similar triangles, in which the like sides are proportional)

(c) State the duties of the leader and follower in chain survey.
(Ans. See Sec. 1.13)

(d) State the purpose of writing inverted figures on the graduated circle of a prismatic compass.
(Ans. The figures are written inverted on the graduated ring so that they can be viewed the right way up while looking through the prism at the time of observation.

(e) State the necessity of providing a ball-and-socket arrangement on the tripod of a prismatic compass.
(Ans. The Ball-and-socket arrangement is provided on top of the tripod for levelling the compass box, as there is no other arrangement for levelling)

(f) State the checks applied on angular measurements taken in a closed compass traverse. (Ans. See Sec. 3.6)

(g) State the different types of levelling operation.
(Ans. See Sec. 5.6)

(h) State the methods of arithmetical check in the calculation of reduced level. (Ans. See Sec. 5.12)

(i) List the methods of plane table surveying. (Ans. See Sec. 4.5)

(j) Explain the term reduced bearing. (Ans. See Sec. 3.2)

(k) State the object of tacheometry. (Ans. See Sec. 11.1)

(l) Give the list of instruments required for route surveying.
(Ans. The following instruments are required for route surveying:
(a) Levelling instrument with stand — 1
(b) Levelling staff — 2
(c) Metric chain (with 10 arrows) — 1
(d) Metallic tape — 1
(e) Plane table with accessories — 1 set

Q. 2 Answer any two:

(a) (i) Explain the method of setting a perpendicular by optical square.
(Ans. See Sec. 2.12)

(ii) Explain the term triangulation. What is the need for obtaining well-conditioned triangles? (Ans. See Sec. 2.1 and 2.2)

(b) (i) The distance between two stations was measured using a 20-m chain, and found to be 1,500 m. The same distance was measured using a 30 m chain, and was found to be 1,497.19 m. If the 20-m chain was 0.05 m too long, what is the error in 30 m chain?
(Ans. + 0.1314 m too long)

(ii) Explain the random line method of crossing an obstacle.
(Ans. See Sec. 1.16)

(c) The following are the bearings observed in a closed traverse survey conducted by using a prismatic compass at a place where local attraction was suspected. At what stations do you suspect local attraction? Find the corrected bearings.

Line	FB	BB
PQ	124°30′	304°30′
QR	68°15′	246°00′
RS	310°30′	135°15′
SP	200°15′	17°45′

(Ans. 0° at P, 0° at Q, + 2°15′ at R, – 2°30′ at S, 124°30′, 68°15′, 312°45′, 197°45′)

Q. 3 Attempt any two:

(a) Draw a neat sketch of a prismatic compass and name the component parts. (Ans. See Sec. 3.8)

(b) Explain the temporary adjustment of a plane table.
(Ans. See Sec. 4.4)

(c) Explain the method of measuring vertical angle by theodolite.
(Ans. See Sec. 9.9)

Q. 4 (a) Give the limits of permissible angular and linear error in theodolite traversing. (Ans. See Sec. 9.17)

(b) The following consecutive readings were observed in one setting along a continuously sloping ground between stations A and B which are 537 m apart; the RL of station A is 100.000:

0.080 on A, 1.365, 2.850, 0.670, 1.990, 1.230, 1.860, 1.770, 2.180, 2.320, 1.010, 1.110 on B.

Calculate: (i) The RLs of all the points, and
(ii) The gradient of the line AB.

(Ans. (i) RLs—98.715, 97.230, 99.410, 98.090, 98.850, 98.220, 98.310, 97.900, 97.760, 99.070, 98.970, and
(ii) Falling gradient = 1/521.36)

(c) Give the characteristics of contour lines. (Ans. See Sec. 6.4)

Q. 5 Attempt any two:

(a) Explain the method of double sighting adopted in prolonging a straight line by theodolite. (Ans. See Sec. 9.13)

(b) The corrected consecutive coordinates of a traverse are given below:

Point	Northing (+)	Southing (–)	Easting (+)	Westing (–)
P	415.20	—	—	175.50
Q	84.60	—	600.70	—
R	—	400.50	101.20	—
S	—	99.30	—	526.00

Calculate the independent coordinates of all the points.

(Ans. From the given coordinates, it is found that R is most southerly station with latitude = – 400.50. S and P are westerly stations with total departure of – 526.40 – 175.50 = – 701.90)

So, let us consider station R as the origin with X-coordinate + 450.00 and Y-coordinate + 750.00.

Station	Independent Y-Coordinate	Independent X-Coordinate
R	+ 450.00	+ 750.00
S	+ 450.00 – 99.30 = + 350.70	+ 750.00 – 526.40 = + 223.60
P	+ 350.70 + 415.20 = + 765.90	+ 223.60 – 175.50 = + 48.10
Q	+ 765.90 + 84.60 = + 850.50	+ 48.10 – 600.70 = + 648.80
R	+ 850.50 – 400.50 = + 450.00	+ 648.80 + 101.20 = + 750.00
	(Checked)	(Checked)

(c) Calculate the data required for setting out a simple circular curve from the following information.

Radius of curve = 300 m, length of long chord = 64 m, interval of ordinate = 8 m.

(Ans. Follow the procedure shown in Sec. 10.6, subsection A)

Q. 6 (a) The following observations were recorded while finding the area of zero circle of a planimeter. What is the required area?

Obs No.	IR	FR	Position of anchor point	Value of N
1	7.980	3.346	Outside the figure	+ 2
2	5.684	8.585	Inside the figure	– 3

(Ans. Area of zero circle = 4,245 cm^2)

(b) What is a clinometer? Describe, how the angle of a slope is measured by mean of this instrument. (Ans. See Sec. 1.15)

(c) Give the list of maps prepared in city surveying. (Ans. See Sec. 12.12)

Appendix C

A.M.I.E. Examinations

SURVEYING

Summer 1990

1. Two ranging rods, one of length 3.00 m and the other of 1.50 m, were used in an effort to find the height of an inaccessible tower. In the first setting, the rods were so placed that their tops were in line with the top of the tower. The distance between the rods was 15 m. In the second setting, the rods were ranged on the same line as before. This time the setting, the rods were ranged on the same line as before. This time the distance between the rods was 30 m. If the distance between the two longer rods was 90 m, find the height of the tower. (Ans. 12 m)
2. The following consecutive readings were taken at 30 m intervals on a continuously sloping ground with a dumpy level and 4 m levelling staff:

   0.680, 1.455, 1.855, 2.330, 2.885, 3.380, 1.055, 1.860, 2.265, 3.540, 0.835, 0.945, 1.530 and 2.250

   The RL of the starting point was 80.750. Calculate the RLs of all the points and find the gradient of the line joining the first point and the last. (Ans. Falling gradient = 1/50)
3. The following tacheometric observations were made with an anallatic telescope having a multiplying constant of 100 on a vertically held staff.

Inst. station	Height of instrument	Staff station	Vertical angle	Stadia readings
P	1.45 m	B.M.	– 6°00′	0°740, 1.860
P	1.45 m	C.P.	– 8°30′	0.620, 1.590
Q	1.40 m	C.P.	– 6°30′	0.595, 1.515

If the RL of the BM is 250.000, find that of station Q. (Ans. 322.76 m)

4. In an effort to layout a pond in a public park, two perpendiculars AD and BC of lengths 40 and 80 m respectively were erected on either side of a line AB of length 240 m. If the pond is to have straight sides lying along AB and DC, the ends being formed of circular arcs to which AB,

DC and the end perpendiculars are tangential, calculate the radii of the two circular areas and the perimeter of the pond.

(Ans. R_1 = 21.67 m
R_2 = 36.69 m
Curve length on one side = 64.5 m
Curve length on other side = 121.32 m
Straight length on one side = 181.64 m
Straight length on other side = 186.64 m
Total perimeter = 549.1 m)

5. In a proposed hydroelectric project, a reservoir was reqired to provide a storage of 4,50 million cubic metres between the lowest draw down (LDD) and the top water level (TWL). The areas contained within the stated contours on upstream faces of the dam were as follows:

   Contour (m)—100–95–90–85–80–75–70–65
   Area (heactores)—30–25–23–17–15–13–7–2

   If the LDD was to be 68, calculate the TWL for full storage capacity. (Ans. 95.27 m)
6. The following notes refer to a closed traverse. Compute the missing quantities:

Line	*Length (m)*	*Bearing*
AB	725	S60°00′E
BC	1050	?
CD	1250	?
DE	950	S55°30′W
EA	575	S02°45′W

(Ans. Bearing of BC = N 63°2′E
Bearing of CD = N 37°3′W)

7. The centre line of a railway track is to be tangential to each of the following lines:

Line	*AB*	*BC*	*CD*
Length	—	990 m	—
Bearing	00°	135°0′	45°

Determine the radius of the curve. (Ans. 700 m)

SURVEYING

Winter 1989

1. A steel tape 30 m long was standardised at 20°C with a pull of 10 kg. Find the true length of the line, if measured length = 312 m, mean temperature during measurement = 35°C pull exerted 16 kg, weight of 1 m^3 of steel = 7,860 kg, weight of tape = 0.8 kg, $E = 2.11 \times 10^{10}$ kg/m^2, and coefficient of expansion of tape per °C = 12×10^{-6}. (Ans. 312.04975 m)

2. The following lengths and bearings of the sides of a traverse PQRST were observed, the length and bearing of QR having been omitted:

Line	Length (m)	Bearing
PQ	725.0	120°15′
QR	?	?
RS	1,250.0	322°24′
ST	945.0	235°18′
TP	577.2	182°40′

Compute the length and bearing of line QR.

(Ans. Length = 1,059.9 m, and Bearing = N 62°30′E)

3. A traverse A B C D was run by a tacheometer fitted with an anallatic lens and having a multiplying constant of 100. The following readings were taken with the staff held normally.

Line	Bearing	Vertical angle	Staff intercept
AB	27°38′	+ 7°4′	1.9
BC	300°24′	+ 4°32′	1.47
CD	236°45′	− 2°10′	1.75

Find the length and bearing of D A.

(Ans. Length = 235.4 m, and Bearing = S52°39′ E)

4. Two tangents intersect at chainage 1,192.0 m, the deflection angle being 50°30′. Calculate the necessary data to set out the required curve by the method of deflection angles. Assume the radius of the curve to be 300 m. The chain length is 20 m.

(Ans. Tangent length = 141.49 m
Curve length = 264.42 m
Ch. of T_1 = 1,050.51 m
Ch. of T_2 = 1,314.93 m
I.S.C. = 9.49 m
No. of full chords of 20 m = 12
FSC = 14.93 m
δ for ISC = 0°54′23′
δ for full chord = 1°54′35″
δ for FSC = 1°25′32″

5. While determining the height of chimney, the following observations were made:

Inst. station	Reading on BM	Angle of elevation	Remark
A	1.266	10°48′	RL of BM
B	1.086	7°12′	= 248.362 m

Points A and B on top of the chimney are in the same vertical plane, and the distance between them is 50 m. What is the elevation of the chimney top? (Ans. 267.777 m)

6. The following staff readings were taken with a level which was shifted after the fourth, seventh and tenth readings.

1.235, 2.005, 1.875, 0.96, 0.38, 1.64, 2.84, 1.75, 1.93, 2.15, 2.37, 2.46

Assuming the RL of the starting point to be 100.000 m, enter the readings as you would on a page of a level book, and determine the RLs of all the points.

(Ans. 99.230, 99.360, 100.275, 99.015, 97.815, 97.635, 97.415, 97.325)

7. The following bearings were observed in running a compass traverse:

Line	FB	BB
AB	38°30′	219°15′
BC	100°45′	278°30′
CD	25°45′	207°15′
DE	325°15′	145°15′

At what stations do you suspect local attraction? Determine the corrected fore and back bearings and the true bearings of each line, if magnetic declination was 10° W.

(Ans. + 1.30′ at A, + 45′ at B, and + 1°30′ at C)

Line	Correct FB	Correct BB	TB
AB	40°00′	220°00′	30°00′
BC	100°00′	280°00′	90°00′
CD	27°15′	207°15′	17°15′
DE	325°15′	145°15′	315°15′

SURVEYING

Summer 1989

1. In chaining an area containing a pond, two points C and D are selected on either side of chain station A such that A, C and D lie on a line. The point B which is on the other side of the pond is on the chain line AB. If the distances AC, AD, BC and BD are 35, 45, 100, and 95 m respectively, determine the length of chain line AB and the angle which the inclined line CD makes with it.

(Ans. Length of AB = 89.43 m, and Inclination of CD with AB = 97°7′57″)

2. A steel tape 20 m long, standardised at 15°C with a pull of 10 kg, is used to measure distance along a slope of 4°25′. If the mean temperature during measurement is 10°C, and the pull applied 16 kg, determine the correction required per tape length. Assume coefficient of expansion = 112×10^{-7} per °C, cross-sectional area of tape = 0.08 cm^2, and $E = 2.1 \times 10^6$ kg/cm^2.

(Ans. − 0.05941 m (too short))

3. The measured lengths and bearings of the side of a closed traverse A B C D E A run counterclockwise, and are tabulated below. Calculate the lengths of CD and DE:

Line	*Length (m)*	*Bearing*
AB	298.7	0°0′
BC	205.7	N25°12′W
CD	?	S75°6′W
DE	?	S56°24′W
EA	213.4	N35°36′E

(Ans. CD = 659.36 m, and DE = 836.87 m)

4. A railway embankment, 500 m long, has a width at formation level of 9 m with side slopes of 2 to 1. The ground levels every 100 m along the centre line are:

Distance (m)—0 – 100 – 200 – 300 – 400 – 500
GL (m)—107.8 – 106.3 – 110.5 – 111.0 – 110.7 – 112.2

The embankment has a rising gradient of 1.2 m per 100 m and the formation level is 110.5 m at zero chainage. Assuming the ground to be level across the centre line, compute the volume of earth work.

(Ans. Volume according to prismoidal formula = 34,803.3 m^3, and Volume according to trapezoidal formula = 32,816 m^3)

5. To ascertain the level difference between A and B, differential levelling was performed from A to B and extended up to C which is a bench-mark having an RL of 95.75 m. Rule out a page of your answer book and determine the level difference between A and B.

Point	BS	IS	FS	Rise	Fall	RL	Remarks
1	1.195						A
2	0.445		2.375				
3	2.150		1.000				
4		0.720					B
5	1.465		0.260				
6	2.630		0.905				
7	2.140		0.975				
8			1.305			95.75	C

(Ans. 0.305 m (fall from A to B))

6. A tacheometer, fitted with an anallatic lens and having a multiplying constant of 100, was set up at R which is an intermediate point on a traverse course AB. The following readings were taken with the staff held vertically.

Staff station	Bearing	Vertical angle	Intercept	Axial hair reading
A	40°35′	– 4°24′	2.21	1.99
B	22°35′	– 5°12′	2.02	1.90

Calculate the length of AB and the level difference between A and B.

(Ans. Length of AB = 68.43 m, and Level difference = 1.237 (fall from A to B))

SURVEYING

Summer 1988

1. A base line was measured to be 150 m long with a tape at a field temperature of 27°C, and the pull applied was 14 kg. The tape was standardised at a temperature of 15°C with a pull of 8 kg. If the designated length of the tape is 20 m, weight of 1 cm^3 of tape = 7.086 kg, weight of tape = 0.8 kg, $E = 2.10 \times 10^6$ kg/cm^2 and coefficient of expansion per °C = 11.2×10^{-6}, find the true length of the line. (Ans. 150.008 m)
2. A fixed hair tacheometer fitted with an anallatic lens, with an instrument constant of 100, was used to determine the slope between two points P and Q. The following observations were made:

Inst. stn.	Staff point	Bearing	Reading of stadia hairs	Reading of axial hair	Vertical angle
A	P	345°	0.915, 2.585	1.750	+ 15°
	Q	75°	0.760, 3.715	2.240	+ 10°

If the staff was held vertically, compute the gradient from P to Q. (Ans. 1 in 39.335)

3. The following bearings were observed in traversing with a compass an area where local attraction was suspected. At what stations do you suspect local attraction? Determine the corrected bearings of the lines.

Line	*FB*	*BB*
PQ	134°30′	314°30′
QR	120°15′	299°20′
RS	3°20′	185°30′
SP	275°	93°45′

(Ans. + 55′ at R, – 1°15′ at S, Correct FBs = 134°30′, 120°15′, 4°15′, 273°45′)

4. The following is the page of a level field book from which several values are missing. Reconstruct the page and fill all the entries marked with a cross. Apply all necessary checks.

Station	BS	IS	FS	Rise	Fall	RL	Remark
1	1.385					100.000	BM
2		1.430			×	×	
3		×			0.395	×	
4	×		1.275	×		×	TP1
5	0.630		0.585	0.310		×	TP2
6		0.920			×	100.13	
7		×			0.210	×	
8			1.740		×	×	

(Ans. BS = 0.895
IS = 1.825, 1.130
Rise = 0.550
Fall = 0.045, 0.290, 0.610
RL = 99.955, 99.560, 100.110, 100.420, 99.920, 99.310)

5. A railway embankment 600 m long has a formation width of 11.5 m with a side slope of 2 to 1. If the ground levels and formation levels are as follows, calculate the volume of earth work. The ground is level across the centre line.

Distance (m)—0 – 100 – 200 – 300 – 400 – 500 – 600
GL (m)—105.2 – 106.8 – 107.0 – 103.4 – 105.6 – 104.7 – 105.1
FL (m)—107.5 – 108.6 – 108.5 – 104.5 – 106.9 – 105.6 – 106.3

(Ans. Prismoidal—11,697.7 m^3, and Trapezoidal—12,115.5 m^3)

SURVEYING
Winter 1988

1. To determine the width of a river, a chain line PQR was laid across it, the point Q and R being on two sides of it. At point S, 60 m away and at right angle to PQ from Q, the bearings of points R and P were measured, and found to be 280° and 190° respectively. If the distance PQ is 32 m, determine the distance QR and draw the relevant sketch.
(Ans. 112.51 m)

2. The bearings and lengths of the sides of a closed traverse A B C D E A are presented below along with the latitudes and departures of known sides. Determine the bearings of AB, and the length of CD.

S.No.	Line	Length (m)	Bearing	Latitudes	Departure
1	AB	725	?	?	?
2	BC	1060	N62°36′	+ 489.45	+ 940.24
3	CD	?	N37°36′W	?	?
4	DE	945	S55°18′W	+ 537.99	– 776.92
5	EA	577.2	S2°40′W	– 576.63	– 26.85

(Ans. Bearing of AB = S59°45′ E, and Length of CD = 1,250 m)

3. The following consecutive readings were taken with a level and 3 m levelling staff on a continuously sloping ground:

0.602, 1.234, 1.860, 2.574, 0.238, 0.914, 1.936, 2.872, 0.568, 1.824, 2.722

Determine the RLs of all the points, if the RL of the first point was 192.122 m.

(Ans. RLs = 191.49, 190.864, 190.150, 189.474, 188.452, 187.516, 186.260, 185.362)

4. The following observations were taken with a tacheometer fitted with an anallatic lens, the staff being held vertically.

Inst. station	Height of axis	Staff station	Vertical angle	Hair readings	Remarks
P	1.45	BM	– 6°12′	0.98, 1.54, 2.10	RL of BM = 384.25
P	1.45	Q	+ 7°5′	0.83, 1.36, 1.89	
Q	1.57	R	+ 12°21′	1.89, 2.48, 30.08	

Determine the distance PQ and QR and the RLs of P, Q and R

(Ans. PQ = 104.39 m,
QR = 114.4 m,
RL of P = 396.365 m,
RL of Q = 409.426 m, and
RL of R = 433.778 m)

5. The following fore and back bearings were observed in traversing an area with a compass:

Line	*FB*	*BB*
PQ	S37°30′E	N37°30′W
QR	S43°15′W	N44°15′W
RS	N73°00′W	S72°15′E
ST	N12°45′E	S13°15′W
TP	N60°00′E	S59°00′W

Calculate the interior angles and correct them for observational errors.

(Ans. Total error = – 25°15′
Correction per angle = + 5°3′
Corrected angles
$\angle P$ = 101°33′,
$\angle Q$ = 104°18′,
$\angle R$ = 122°18′,
$\angle S$ = 100°3′, and
$\angle T$ = 111°48′)

6. To compute the volume of earth work between two stations 60 m apart, the following notes were taken for the three-level cross-sections at two stations:

Station	Cross-section		
A	$\frac{1.7}{7.7}$	$\frac{2.6}{0}$	$\frac{4.5}{10.8}$
B	$\frac{2.6}{8.5}$	$\frac{3.6}{0}$	$\frac{6.5}{12.3}$

If the width of cutting at the formation level is 10.5 m, calculate the volume of cutting between the two stations. (Ans. 3,008.25 m^3)

SURVEYING

Winter 1987

1. A survey line ABC cuts the banks of a river at B and C. To determine the distance BC, a line BE, 50 m long was set out roughly parallel to the river. A point D was then found in CE produced and the middle point F of DB determined. FE was then produced to G, making FG equal to EF, and DG produced to cut the survey line at H. GH and HB were found to be 33.33 and 66.67 m long respectively. Calculate the distance BC.
(Ans. 100 m)
2. A steel tape 20 m long standardised at 55°F with a pull of 10 kg was used for measuring a base line. Find the correction per tape length if the temperature at the line of measurement was 80°F and the pull exerted 15 kg. Weight per cubic centimetre of steel = 7.86 g, weight of tape = 0.80 kg, $E = 2.11 \times 10^6$ kg/cm^2, coefficient of expansion of tape per 1°F $= 6.109 \times 10^{-6}$. (Ans. + 0.0016 m too long)
3. A road bend which deflects by an angle of 80° is to be designed for a maximum speed of 120 km/h, a maximum centrifugal ratio of 1/4, and a maximum rate of change of acceleration of 30 cm/s^3, the curve consisting of a circular arc combined with two cubic spirals. Calculate: (i) the radius of the circular curve, (ii) the requisite length of the transition curve, and (iii) the total length of the composite curve.
(Ans. (i) 453.05 m, (ii) 272.42 m, and (iii) 905.51 m)
4. To find the elevation of the top Q of a hill, a flagstaff of height 3 m was erected and observations were made from two stations P and R, 160 m apart. The horizontal angle measured at P between R and the top of the flagstaff was 60°30′ and the angle measured at R between the top of the flagstaff and P was 68°18′. The angle of elevation to the top of the flagstaff was measured to be 10°12′ at P, and 10°48′ at R. The staff reading on the BM when the instrument was at P was 1.865 m and that with the instrument at R was 2.055 m. Calculate the elevation of the top of the hill if that of the BM was 135.00 m.
(Ans. RL of Q (top of hill) = 168.16 m)
5. In the course of a tacheometric survey, the following observations were made for the normal cross-section of a stream, the instrument being set up on the bank with a telescope level. Assume a multiplying constant of 100.

Staff point	Stadia readings			Remarks
1	4.62	4.76	4.90	Edge of water
2	6.06	6.27	6.48	
3	7.25	7.63	8.01	
4	8.30	8.32	9.34	
5	8.36	8.96	9.57	
6	7.31	8.05	8.80	
7	5.67	6.54	7.41	
8	3.69	4.76	5.81	Edge of water

Draw the cross-section of the river and find the cross-sectional area.

(Ans. Water level at point 1 = 0 (edge of water)
Water level at point 2 = 1.51 m
Water level at point 3 = 2.87 m
Water level at point 4 = 3.56 m
Water level at point 5 = 4.20 m
Water level at point 6 = 3.29 m
Water level at point 7 = 1.78 m
Water level at point 8 = 0 (edge of water)
Distance between sections 1 and 2 = 14 m
Distance between sections 2 and 3 = 34 m
Distance between sections 3 and 4 = 28 m
Distance between sections 4 and 5 = 17 m
Distance between sections 5 and 6 = 28 m
Distance between sections 6 and 7 = 25 m
Distance between sections 7 and 8 = 38 m
Draw the cross-section yourself.
Cross-sectional area = 443.065 m^2)

The width of cutting at the formation level is 12 m. Calculate the volume of cutting between the two stations using the trapezoidal formula.
(Ans. 3,029.625 m^3)

SURVEYING

Summer 1987

The area of the plan of a survey plotted to a scale of 10 m to 1 cm measures now as 400.25 cm^2 as determined by planimeter. The plan is found to have shrunk so that a line originally 10 cm long now measures only 9.75 cm. There was also a note on the plan that the 20 m chain used was 7.5 cm too short. Find the true area of the survey.
(Ans. 41,945.99 m^2)

2. The following consecutive readings were taken with a level and 3 m levelling staff on continuously sloping ground at common intervals of 20 m:

0.625, 1.235, 1.860, 2.575, 0.240, 0.915, 1.935, 2.875, 0.570, 1.825, 2.725

The reduced level of the first point was 192.120. Rule out a page of a level book and enter the above readings. Calculate the RLs of all the points and also the gradient of the line joining the first point and the last.

$\left(\text{Ans. Falling gradient } \frac{1}{23.67}\right)$

3. The following table gives the corrected latitudes and departures (in metres of the side of a closed traverse PQRS.

Side	Latitude		Departure	
	N	S	E	W
PQ	216	—	8	—
QR	30	—	498	—
RS	—	246	8	—
SP	0	—	—	514

Compute its area by: (i) the departure and total latitude method, and (ii) the coordinate method.

(Ans. (i) 116,886 m^2, and (ii) 116,886 m^2)

4. The following bearings were observed with a compass. Calculate the interior angles:

Line		FB
JM	—	56°30′
MN	—	118°00′
NQ	—	42°00′
OP	—	201°30′
PL	—	296°00′

(Ans. ∠L = 59°30′, ∠M = 118°30′, ∠N = 256°00′, ∠O = 20°30′, and ∠P = 85°30′)

5. The vertical angles to vanes fixed 1 and 3 m above the foot of a staff held vertically at a station A were 3°10′ and 5°24′ respectively. Find the horizontal distance and reduced level of A, if the height of the instrument axis is 138.556 m above datum.

(Ans. Horizontal distance = 51.02 m, RL of A = 140.377 m)

6. A transition curve is required for a circular curve of radius 300 m, the gauge being 1.5 m and maximum superelevation restricted to 12.5 cm. The transition curve is to be designed for a velocity such that no lateral pressure is imposed on the rails and the rate of gain of radical acceleration is 25 cm/s^3. Calculate the required length of the transition curve and the design speed.

(Ans. Length of transition curve = 51.205 m, Speed = 56.376 km/hr)

7. The following notes refer to three level cross-sections at two stations 50 m apart:

Station	Cross-section		
1	$\frac{1.9}{7.7}$	$\frac{3.0}{0}$	$\frac{4.8}{10.8}$
2	$\frac{3.1}{9.1}$	$\frac{3.9}{0}$	$\frac{7.1}{13.1}$

The width of cutting at formation level is 12 m. Calculate the volume of cutting between the two stations using trapezoidal formula.

(Ans. 3029.625 m^3)

SURVEYING

Winter 1986

1. A and B are two points, 200 m apart, along the south bank of a river, flowing due east-west. The magnetic bearings of a tower situated on the opposite bank, as observed from A and B, are 40° and 310° respectively. Determine the width of the river. (Ans. 98.47 m)

2. The following perpendicular offsets were taken at 20 m intervals from a base line to an irregular boundary line:

5.9, 12.4, 16.5, 15.3, 18.4, 20.9, 24.2, 21.8, and 19.2 m

Calculate the area enclosed between the two end offsets, the base line and the irregular boundary line by: (i) the average ordinate rule, (ii) the trapezoidal rule and (iii) Simpson's rule.

(Ans. (i) 2,748.44 m^2, (ii) 2,841.0 m^2, and (iii) 2,832.67 m^2)

3. The magnetic bearing of a line AC is 137°45′. Station B is situated on the western side of the line AC. The angles A, B and C of the triangle are 67°15′, 71°45′, and 41°00′ respectively. Calculate the fore and back bearings of all the three sides of the triangle. If the magnetic declination in the area is 2°30′W, determine the true bearings of all the sides of the triangle.

(Ans.

Line	FB	BB	TB
AB	205°00′	25°00′	202°30′
BC	96°45′	276°45′	94°15′
CA	317°45′	137°45′	315°15′

4. The areas enclosed by the contour lines of a reservoir, at 5 m intervals, up to the face of the proposed dam, are as shown below:

Value of contour (m)— 1,005 – 1,010 – 1,015 – 1,020 – 1,025 – 1,030 – 1,035

Area (m^2)— 400 – 1,500 – 3,000 – 8,000 – 18,000 – 25,000 – 40,000

Taking 1,005 and 1,035 m as the bottom-most and the highest water level, respectively, of the reservoir, determine the capacity of the reservoir by: (i) the trapezoidal formula, and (ii) the prismoidal formula.

(Ans. (i) 378,500 m^3, and (ii) 367,333.33 m^3)

5. The following readings were taken on a vertical staff with a stadia tacheometer fitted with an anallatic lens and having a multiplying constant of 100.

Staff stn	Bearing	Stadia readings	Vertical angle
P	47°10′	0.940, 1.500, 2.000	+ 8°00′
Q	227°10′	0.847, 2.000, 3.153	– 5°00′

Determine the height of the ground between P and Q, and the gradient of the slope from P to Q.

(Ans. Height of ground between P and Q = 35.958 m (fall from A to B), and Falling gradient of PQ = 1/9.42)

6. The following figures were extracted from a level book, some of the entries being illegible (marked here with crosses). Insert the missing figures, check your results, and rebook all the figures using the 'rise and fall' method:

Stn	BS	IS	FS	Rise	Fall	RL	Remark
1	2.285					232.460	BM 1
2	1.650		×	0.020			
3		2.105			×		
4	×		1.960	×			
5	2.050		1.925		0.300		
6		×		×		232.255	BM 2
7	1.690		×	0.340			
8	2.865		2.100		×		
9			×	×		233.425	BM 3

(Ans. BS = 1.625,
IS = 1.665,
FS = 2.265, 1.325, 1.625,
Rise = 0.145, 0.385, 1.240, and
Fall = 0.455, 0.410)

7. A and B are two stations of an open traverse and their independent coordinates are as follows:

Station	Latitude (m)	Departure (m)
A	27,456.8	6,007.2
B	26,936.0	7,721.6

It is proposed to construct a railway track from C, roughly south of A to D, roughly north of B. While C and D are not intervisible, the perpendicular offsets from the traverse stations to the railway track are measured to be AC = 104 m and BD = 57.6 m. Determine the WCB of CD.

(Ans. 101°44′38″)

SURVEYING

Summer 1986

1. In running fly levels from a benchmark of RL 183.185, the following readings were obtained:

BS—2.085, 1.025, 1.890, 0.625
FS—1.925, 2.820, 0.890

From the last position of the instrument, five pegs at 25 m intervals are to be set out on a uniformly falling gradient of 1 in 100, with the first peg having an RL of 182.350. Determine the staff readings required for setting the tops of the five pegs on the given gradient.

(Ans. 0.825, 1.075, 1.325, 1.575, 1.825)

2. Find the RL of the top of a chimney from the following data:

Inst. stn	Reading on BM	Angle of elevation	Remarks
A	1.265	18°34′20″	RL of BM = 242.830 m
B	1.625	10°12′40″	Distance AB = 60 m

Stations A and B and the top of the chimney are in the same vertical plane.

(Ans. 267.335 m)

3. The following observations were taken from traverse stations A and B to points C and D by means of a stadia tacheometer fitted with an anallatic lens, the instrument constant being 100:

Inst. stn	Staff station	Height of inst.	Bearing	Vertical angle	Staff reading
A	C	1.48	126°30′	+ 12°30′	0.770, 1.600, 2.430
B	D	1.42	184°45′	– 10°30′	0.860, 1.840, 2.820

Coordinates of station A = 112.8N, 106.4W
Coordinates of station B = 198.5N, 292.6W
Determine the length of line CD.

(Ans. 329.52 m)

4. The following bearings were observed in a compass traverse:

Line	FB	BB
AB	305°00′	125°30′
BC	75°30′	254°30′
CD	115°30′	297°00′
DE	165°30′	345°30′
EA	225°00′	44°00′

At what stations is local attraction suspected? Find the corrected MB. If declination was 4°30′W, determine the true bearing.

(Ans. +1° at A, +30′ at B, and +1°30′ at C,

Line	MB	TB
AB	306°00′	301°30′
BC	76°00′	71°30′
CD	117°00′	112°30′
DE	165°30′	161°00′
EA	225°00′	220°30′)

5. Two straights T_1 P and P T_2 are intersected by a third line AB, such that ∠PAB = 46°24′, ∠PBA = 32°36′ and distance AB = 312 m. Calculate the radius of the simple circular curve which will be tangential to the three lines T_1 P, AB and P T_2, and the chainages of the point of curve (T_1) and point of tangency (T_2), if the chainage of the point P is 2,857.5 m.

(Ans. R = 432.68 m,
chainage of T_1 = 2,500.8 m, and
chainage of T_2 = 2,933.48 m)

6. The following perpendicular offsets were taken from a chain line to a barbed wire fence:

Chainage (m)—0 – 20 – 40 – 60 – 80 – 95 – 110 – 140 – 170 – 200
Offsets (m)—6.7 – 5.8 – 10.3 – 12.8 – 9.7 – 8.8 – 6.9 – 8.2 – 6.5 – 5.8

Calculate the area between the chain line, the barbed wire fence and the end offsets by: (i) the trapezoidal rule, and (ii) Simpson's rule.

(Ans. (i) 1,630 m^2, and (ii) 1,648.17 m^2)

7. A road embankment 500 m long is 15 m wide at the formation level and has a side slope of 2 : 1. The ground levels at every 100 m along the centre line are as follows:

Distance (m)—0 – 100 – 200 – 300 – 400 – 500
GL (m)—105.2 – 106.5 – 107.6 – 107.2 – 108.3 – 108.8

The formation level at zero chainage is 107.0 m and the embankment has a rising gradient of 1 : 100, while the ground is level across the centre line. Calculate the volume of earth work according to the prismoidal rule. (Ans. 25,754 m^3)

Appendix D

Technical Examinations (Polytechnics) Andhra Pradesh

SURVEYING I, DECEMBER 1991

Part A

1. State the fundamental principles of surveying. (Ans. See Sec. 1.5)
2. List any four field signals in chain surveying practice, along with their respective meanings.

(Ans. During ranging the following signals are given by the ranger to his assistant:

Signal 1—When only the right hand is waved gently, it indicates that the ranging rod is to be moved to the right very slowly.
Signal 2—When only the left hand is waved, it indicates that the rod is to be moved to the left.
Signal 3—When both hands are moved up and down, it indicates that ranging is perfect, and that the rod is to be fixed in position.
Signal 4—When the ranger stoops, it indicates that the verticality of the rods has to be checked.

3. Sketch the conventional signs of:
(a) An embankment, (b) a double railway line,
(c) a road bridge, and (d) a cart track
(Ans. See Sec. 2.10)
4. Explain: (a) Quadrantal bearing, (b) declination, (c) magnetic bearing, and (d) local attraction. (Ans. See Sec. 3.2)
5. In an old survey made when the declination was 2°30′ West, the magnetic bearing of a line was 215°. The declination in the same locality is now 3°45′ East. What are the true and present magnetic bearings of the line?
(Ans. True bearing = 212°30′, and Magnetic bearing = 208°45′)
6. Convert the following whole circle bearings to quadrantal bearings:

Line	Bearing
AB	171°30′
BC	38°00′
CD	256°00′
DA	328°30′

(Ans. AB = S8°30′E, BC = N38°00′E,
CD = S76°00′W, DA = N31°30′W)

7. Define the terms:
(a) Benchmark, (b) level surface,
(c) line of collimation, and (d) vertical axis.
(Ans. See Sec. 5.2)

8. State the conditions of adjustment between the fundamental lines of a dumpy level. (Ans. See Sec. 5.20)

9. List the steps involved in performing the temporary adjustment of a levelling instrument. (Ans. See Sec. 5.5)

10. In a fly levelling operation, the backsight reading on the BM is 0.865, and its RL is 150.420. The staff is held inverted touching the bottom of a sunshade, and the reading taken is 2.345. Calculate the RL of the bottom of the sunshade. (Ans. 153.630 m)

Part B

11. (a) Explain the method of ranging when high ground intervenes between two stations. (Ans. See Sec. 1.8)
(b) Explain the method of continuing chaining when the chain line crosses a pond. (Ans. See Sec. 1.16)

12. (a) A certain field was measured with a 20 m chain and found to have an area of 60 m^2. It was afterwards found that the chain was 0.1 m too short. What is the true area of the field? (Ans. 59.4015 m^2)
(b) The following offsets were taken at 5 m intervals from a chain line to an irregular boundary:

2.8, 3.3, 3.5, 2.7, 2.6, 2.5, 1.9, 2.9, 3.8 m

Using Simpson's rule, determine the area enclosed between the chain line, the boundary line and the end offsets.
(Ans. Area = 113.67 m^2)

13. The following bearings were observed in a compass traverse affected by magnetic attraction:

Line	FB	BB
PQ	72°45′	252°00′
QR	349°00′	167°15′
RS	298°30′	118°30′
ST	229°00′	48°00′
TP	135°30′	319°00′

At what stations do you suspect local attraction?
Correct the bearings for local attraction.
(Ans. 2°30′ at P, – 1°45′ at Q, and + 1°0′ at T; 70°15′, 347°15′, 298°30′, 229°00′, and 136°30′.)

14. (a) Explain four situations which are favourable and two unfavourable for conducting compass survey.
(Ans. Favourable situations:
1. Large area,
2. Area crowded with many details where triangulation is not possible,
3. No sources of local attraction, and
4. Fairly level ground.

Unfavourable situations:
1. Area having plenty of magnetic substances like steel structures, iron ore, electric cables conveying current, etc.
2. In heavy winds, when measurement of bearing becomes difficult

(b) Explain the Bowditch rule for adjusting a compass traverse.
(Ans. See Sec. 3.17)

15. A closed traverse ABCDE is run clockwise and the following interior angles are observed:

$\angle A = 68°27'$, $\angle B = 134°47'$, $\angle C = 148°24'$
$\angle D = 59°34'$, and $\angle E = 128°48'$

If the forward bearing of the line AB is 10°12′, calculate the FBs and BBs of all the lines.

(Ans.

Line	FB	BB
BC	55°25′	235°25′
CD	87°1′	267°1′
DE	207°27′	27°27′
EA	258°39′	78°39′

16. A page of a level field book partly defaced is reproduced below. Supply the missing entries marked with a cross.

	BS	IS	FS	Rise	Fall	RL
	1.200					100.00
		×			0.650	×
	1.350		1.565	×		×
		×		0.085		×
	×		×		×	×
		×		×		100.240
			2.250		×	99.150
Total	4.415		×	1.075	×	

(Ans. BS = 1.865
IS = 1.850, 1.265, 1.160)

FS = 1.450
Rise = 0.285, 0.705
Fall = 0.185, 1.090
RLs = 99.350, 99.635, 99.720, 99.535
ΣFS = 5.265
ΣFall = 1.925)

17. Describe the method of conducting field work for profile levelling for a road project. (Ans. See Sec. 5.14)
18. The following reciprocal observations were made with one level.

Instrument at	Staff reading on		Remarks
	P	Q	
P	1.210	2.545	Distance PQ = 1,315 m
Q	0.580	1.985	RL of Q = 532.130 m

Determine; (a) the RL of P, (b) the combined correction for curvature and refraction, and (c) the error in the collimation adjustment of the instrument.

(Ans. (a) 533.566 m, (b) 0.1164 m, and (c) collimation error per 100 m = – 0.0165 (negative))

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