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THE GLASGOW TEXT BOOKS OF CIVIL ENGINEERING. EDITED BY G. MONCUR, B.Sc. M.I.C.E. *Professor of Civil Engineering in the Royal Technical College, Glasgow.*

SURVEYING & FIELD WORK

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THE GLASGOW TEXT BOOKS.
EDITED BY G. MONCUR.

SURVEYING & FIELD WORK

A PRACTICAL TEXT-BOOK ON SURVEYING LEVELLING & SETTING-OUT

INTENDED FOR THE USE OF STUDENTS IN TECHNICAL SCHOOLS AND COLLEGES AND AS A WORK OF REFERENCE FOR SURVEYORS, ENGINEERS AND ARCHITECTS

BY
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PREFACE

THE aim of the author in the preparation of this work has been to produce a thoroughly sound text-book on the principles and practice of surveying, including levelling and setting-out, which, while being primarily intended for the use of students in technical schools and colleges, would at the same time be of value as a work of reference for the surveyor, engineer and architect.

In the arrangement of the work the time-honoured plan of beginning with chain surveying, and making it the vehicle for a comprehensive description of the elements of field work and office work, has been adhered to. While it may appear to some that too great importance is thereby attached to chain surveying, the arrangement has the advantage of taking the beginner at once, in the first eight chapters of the book, through all the processes which go to the attainment (by particular methods) of the ultimate purpose of surveying, namely, the finished survey plan. The various methods of surveying involving measurement of angles are dealt with in subsequent chapters, the order of consideration in each case being first the field work, then the office work. Traverse surveying with the theodolite has been somewhat fully dealt with, as being probably the method in most general use for surveying limited areas, and very full consideration has been given to the co-ordinate or latitude and departure method of plotting traverse surveys. The subject of minor triangulation has been included because of its importance in the case of surveys which extend over larger areas, but the subjects of tacheometry, plane tabling, geodetic surveying and astronomical work, and the more difficult branches of levelling and setting-out, have not been dealt with in this volume.

Endeavour has been made, by means of worked-out examples, and the employment of over 270 diagrams, all specially prepared for this work, to make the explanations of principles and methods and descriptions of instruments clear and explicit. In regard to

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field work and surveying practice, and the office work of preparing plans, numerous practical hints have been given, drawn from a wide experience, and attention has been called to many of the pitfalls which beset the beginner.

The necessity for accuracy and precision in all surveying operations and the importance of systematic routine and proper checking have been emphasised throughout. Absolute accuracy in the measurement of lengths and angles being, however, out of the question, the aim in practical surveying, where the time and money available for the work are usually both strictly limited, should be to attain in the most economical manner at least that degree of precision which is necessary for the purpose of the survey. Precision in surveying is attained by a process of cutting down errors, and, in the author's opinion, it is only from a careful appreciation of the relative magnitudes of the errors which may arise from various sources in the operations of surveying and levelling that a surveyor is enabled to expend his care and effort to the best advantage. For this reason the subject of errors has throughout received detailed treatment.

The author's acknowledgments are due to Mr. Guthrie Brown for valuable assistance in the preparation of the diagrams, to Messrs. Cooke for the loan of the block from which Fig. 127 was prepared, and to many authors in the field of surveying for suggestions.

JAMES WILLIAMSON.

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SURVEYING

CHAPTER I

SURVEYING—FUNDAMENTAL PRINCIPLES

THIS chapter deals with the purpose of surveying and the simple geometrical principles, involving measurements of lengths and angles, on which it is based. The nature of the general problem, having regard to the fact that the earth's surface is not plane but curved, is treated briefly, and indication is given as to when the effect of curvature may be neglected.

Use of Survey Plans.—For many purposes connected with the work of the land surveyor, architect and civil engineer, such as the calculation of areas of ground, recording of boundaries, designing and laying out of buildings, roads, railways and other works, it is necessary to have a plan of the portion of ground concerned showing in accurate proportion the principal features of the surface.

Methods of Representing Surface of Ground on Reduced Scale.—If a portion of ground is not level, a complete representation of its surface on a small scale could only be given by means of a model constructed to show accurately the varying altitudes of the surface as well as the features that are evident in a horizontal projection. Such a model of a portion of ground is occasionally found useful for particular purposes. In ordinary work, however, it is the practice to represent the features of the ground by means of lines and conventional markings drawn on a flat surface, usually a sheet of paper. On such a surface it is possible to show directly only a horizontal projection of the features of the land, and this is what any ordinary survey plan shows.

Plane Surveying.—Plane surveying denotes the methods and procedure which are applicable to surveying an area of small extent, where the earth's surface can be assumed to be a horizontal plane

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and the effect of curvature is negligible. Plane surveying only will be dealt with in this book, but the effect of curvature will be investigated briefly and indication given as to when the error involved in neglecting it becomes practically appreciable.

"Surveying": What it Includes.—Surveying comprises all the operations necessary for the determining and recording on the plan of all the features of the surface of the ground, features of altitude, as well as those such as boundaries, streams, houses, &c., which are ordinarily shown in horizontal projection. It includes the taking of a connected series of measurements on the ground, so arranged as to fix the true relationship of the features of the ground to each other, the laying down of these measurements in correct proportion on paper, and the representing therefrom of the boundaries, streams, houses, &c., by means of lines drawn in ink.

Under surveying we may also properly include the "setting out of works," as exactly the same principles are involved, but the operations are carried out in the reverse order. In this case the outlines and forms of intended works, such as buildings, roads, railways, &c., having been devised and drawn to scale on a plan, require to be marked off full-size on the ground to enable construction to proceed.

As survey plans are commonly used for the purpose of ascertaining areas of land and quantities of earthwork in connection with estate, architectural and engineering work, it will be expedient to consider the principles of making the computations for the above purposes as falling directly under surveying.

Surveying based on Simple Geometrical Principles.—The practice of surveying is based directly on very simple geometrical principles involving the measurements of lengths and angles, and, fundamentally, has to deal with the fixing of the position of a point on the ground in relation to *two* other points whose relative positions are already known. The following four methods of doing this are in common use in surveying:—

(1) By one linear measurement when the points lie in one straight line.

(2) By two linear measurements.

(3) By one linear and one angular measurement.

(4) By two angular measurements.

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Method by One Linear Measurement.—The first method is applicable where the point to be fixed lies on the line joining the two known points, or on the extension of that line. In this case a single measurement of length suffices to determine the point. Let A and B (Fig. 1) represent the two known points, and let C be a point lying on the line joining AB, and D a point lying on the extension of that line. Point C would be fixed with reference to A and B by measuring either the length AC or the length BC; similarly a measurement of the length BD fixes the point D. Points

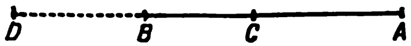

FIG. 1.

C and D would be plotted on the paper by marking off to the proper scale, along the straight line drawn through the points A and B, the measured lengths of AC and BD. It is evident that the positions of any number of points may be fixed along a straight line by this method, which is exemplified in the ordinary procedure of chaining a survey line, as described in Chapters III. and IV.

Method by Two Linear Measurements.—Let A and B (Fig. 2) represent the two known points, and let C be the point whose position is required to be fixed with reference to points A and B, and assume also that all lie in one horizontal plane. To fix point C by method No. 2 it is necessary to measure on the ground the straight horizontal lengths from A to C and from B to C. The distance from A to B is, of course, already known. To plot the points on paper, first mark off points A and B at the correct distance apart by scale. Then, with a pair of compasses, sweep intersecting arcs from A and B as centres with radii equal to the lengths of AC and BC respectively, to the scale of the plan. The point of intersection of the two arcs represents approximately the position of point C with reference to points A and B, the degree of approxi-

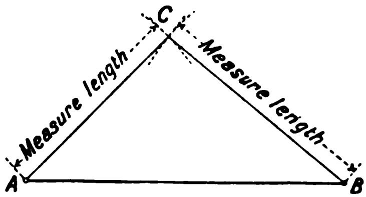

FIG. 2.—Fixing a Point. Method No. 2.

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mation depending on the accuracy with which the various lengths have been measured and plotted. With perfect accuracy throughout, the triangle ABC on the paper would be an exact representation in miniature of the triangle formed by the three points on the ground. A point having been determined, as above described, by this method, it may then be used as a known point from which to fix others. Thus, in Fig. 2, point C having been fixed, the points A and C may be used as known points from which to fix another point, D, and so on successively. This is the principle on which the main points and lines are determined in *Chain Surveying.*

Method by One Linear and One Angular Measurement.—To fix point C by method No. 3, as shown in Fig. 3, it is necessary to measure the straight horizontal length between the points A and C,

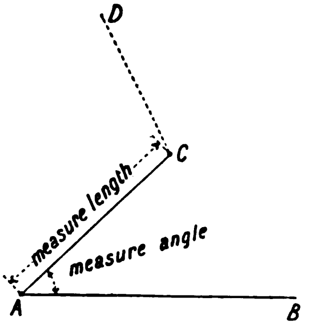

FIG. 3.—Fixing a Point.
Method No. 3.

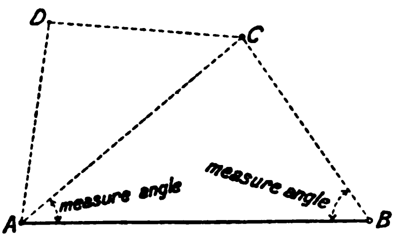

FIG. 4.—Fixing a Point.
Method No. 4.

and the horizontal angle which the direction of the straight line AC makes with the direction of the straight line AB. The similar measurements made from point B, namely, the length BC and the angle at B, would, of course, equally suffice to fix point C. To plot point C on paper a protractor may be used to lay off through point A a straight line making the observed angle with the line AB. The observed length of AC being then marked off to scale along this line, the point C is fixed. Other methods of plotting point C may, however, be used. For example, the length from B to C may be calculated by the principles of trigonometry, and point C may then be plotted as described under method No. 2.

This method may be extended to the fixing successively of any number of points. Thus, a point D may be fixed with reference to

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C and A by measuring the length CD and the angle which CD makes with CA. This is the principle on which *Traverse Surveying* is based.

Method by Two Angular Measurements.—In method No. 4, illustrated in Fig. 4, the measurements to be taken on the ground are the magnitudes of the horizontal angles which the directions AC and BC make with the direction AB. Point C could evidently then be plotted by repeating those angles on the paper, that is, by laying off with the aid of a protractor a straight line through point A making an angle with AB equal to the corresponding angle measured on the ground, and by laying off through B a straight line making with BA an angle equal to the observed angle at B. The intersection of these two straight lines would determine point C. In this case also, point C may be plotted by other methods, the usual procedure being to calculate the lengths AC and BC by the principles of trigonometry and then to plot the point by method No. 2. Method No. 4 can, like the other methods, be applied successively to the determination of any number of points. Thus, point D may be fixed with reference to A and C by observing the angles which the directions AD and CD make with the now known direction AC. This is the principle on which the main reference points are determined for most surveys of large extent. It is known as *Triangulation.*

Methods applied to determining Altitudes of Points.—It might on first consideration seem that the principles involved in determining the altitudes of points above a fixed horizontal plane would necessarily be different from those involved in determining the relationship of points in a horizontal plane. The principles are, however, exactly the same, and are practically limited to methods Nos. 1, 3 and 4 already described, although method No. 2 is of quite possible application.

Method No. 1 applied to the determining of relative altitudes is represented by the process of ordinary levelling by means of a levelling instrument. In this case successive horizontal planes are given by the line of sight of the instrument when placed in different positions, and the vertical distances between the planes are deduced from the readings of the line of sight on a graduated rod or staff held vertically as described in Chapter XVI.

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Corresponding to method No. 3 for fixing points in a horizontal plane, we have the method, illustrated in Fig. 5, for determining the vertical difference in height between two points. A and B are two points on uniformly sloping ground. The measurements required on the ground are the straight distance from A to B measured along the slope and the vertical angle made between the direction AB and a horizontal line AC in the same vertical plane as AB. The vertical height between the points can be arrived at by plotting the line AB in relation to a horizontal line AC, but in practice would usually be got by calculation. See Chapter XV.

The determining of the altitude of a point by method No. 4 is illustrated in Fig. 186, Chapter XV.

Effect of Curvature of Earth.—In what precedes, the surface of reference has been intentionally limited to a horizontal plane. As,

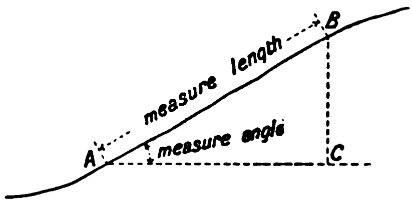

FIG. 5.—Measurement of Altitude.
Method No. 3.

however, the surface of the earth, apart from the irregularities of hills, valleys, &c., is a curved surface, it would not be possible to represent accurately on sheets of paper the features of any considerable area on the assumption that the surface was a plane. In dealing with any large extent of ground the surface of reference is taken as the ideal surface formed by the mean level of the sea assumed as continued and completed right round the globe.

Fig. 6 represents a section taken through the centre of the earth and a portion of its surface, the curvature of the earth and height of the ground being enormously exaggerated. Point O represents the centre of the earth, and A and B are two points on the surface of the ground whose distance apart it is required to find, referred to mean sea level. The line AO joining point A to the centre of the earth represents the vertical direction at point A. Similarly line BO represents the vertical direction at point B. It must be clearly kept in mind that the vertical directions at two different points on the earth's surface, even when the points are quite close together, are never parallel. For two points which are one geographical mile apart the angular inclination between the verticals is one minute. For points 100 ft. apart the inclination of the verticals is roughly one second. In the figure point A′ represents the projection of point A at mean sea level, and B′ is the projection of point B. The distance between the points A and B referred to mean sea level is then the length A′ B′ measured along the curved line marked mean sea level. It is evident that this length is less than the level distance B*b* between the points measured at the level of B, and still less than the level distance A*a* measured at the level of point A. The extent of the difference, however, is not great for moderate difference of altitude. Let the vertical height B′B be one mile and take it that the earth's semi-diameter is roughly 4,000 miles, then the length B*b* would be greater than the length A′B′ by $\frac{1}{4000}$th part. This extent of error might or might not be negligible, depending on the purpose of the survey, but in what follows it will be assumed that for ordinary purposes and moderate altitudes the lengths need not be reduced to correspond with mean sea level.

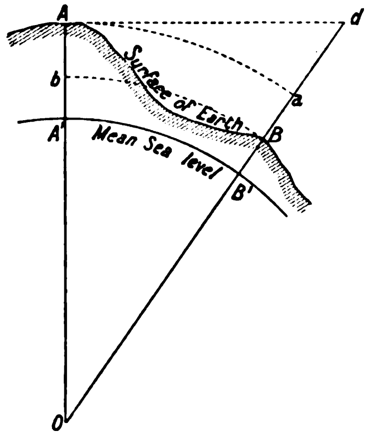

FIG. 6.—Curvature of Earth.

As to neglecting Curvature of the Earth.—When a survey is of small extent also, the further assumption will be made that the surface of the earth is plane, no account being taken of its curvature. A survey may be considered as of small extent when its dimensions do not exceed ten miles by ten miles. A circular portion of the earth's surface ten miles in diameter has approximately the form of a circular curved watch glass, and is dished to the extent of 17 ft. in the centre. The differences between any lengths or areas measured on such a surface and the corresponding lengths or areas measured on a plane surface ten miles in diameter are

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almost inappreciable. For an area 100 miles in diameter, which is dished nearly 1,700 ft. in the centre, the curvature of the surface would require to be taken into account.

Curvature of the Earth to be considered in Finding Altitudes.—The foregoing considerations refer only to the curvature of the earth so far as affecting the making of a plan showing a horizontal projection of the surface features. In the operations required for the determining of altitudes the effect of curvature of the earth's surface becomes appreciable at very much smaller distances. Referring again to Fig. 6, suppose that a levelling instrument is set up at A

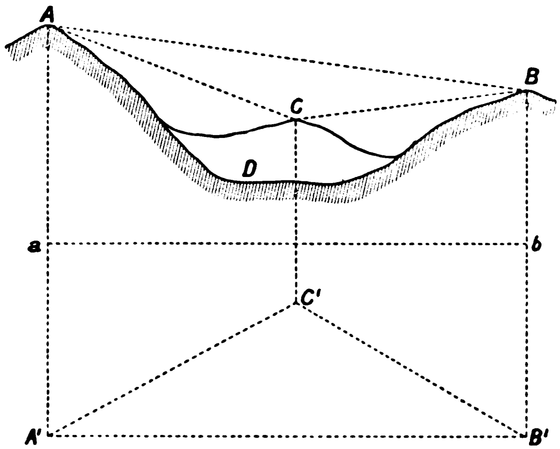

FIG. 7.—Measuring Horizontal Projections of Lengths and Angles.

so as to give a line of sight A*d* perpendicular to the vertical at A, that is, the line A*d* is a horizontal line at the point A. Let *d* be the point where it strikes the vertical through B. Then *d*B does not give the true difference in height between the points A and B, but, due to the curvature of the earth, it is greater than the true height by the amount *da*. At a distance of one mile from A the distance *da* representing the deviation of the earth's surface due to curvature would be 8 ins. approximately. The deviation varies as the square of the distance from A, so that at one-eighth of a mile it would amount to $\frac{1}{8}$ in. Therefore, the horizontal line of sight of a levelling instrument may only be taken as defining a level surface within about a radius of 200 yards if the error is limited to $\frac{1}{8}$ in.

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Horizontal Projections of Lengths and Angles to be Measured.—In describing the methods of fixing the relationship of three points in plan, the assumption was made that all three points lay in one horizontal plane, on level ground. When the points do not lie in one horizontal plane the distance between the points measured along the surface of the ground will not be the proper distance for use in plotting a horizontal projection of the points. Let A and B (Fig. 7) be two points on uneven ground, and let C be the elevation of a third point on a hill some distance beyond the vertical plane through A and B. Let A′, B′, C′, be the projections of the three points on a horizontal plane. Then in measuring from A to B the distance to be determined is not the length along the ground surface ADB, nor yet the direct distance AB, but the horizontal projection of the latter, that is, length *ab* or A′B′. Similarly in measuring angles. What is required is not the actual angle between two lines in a plane containing the lines, but the horizontal angle made by the projections of the lines on a horizontal plane. Thus in measuring for surveying purposes the angle between the lines joining point C to points A and B we desire to find, not the angle made by these lines in the sloping plane containing them, but the angle A′C′B′ made by the projections of these lines on a horizontal plane.

Difficulties of Surveying not in Principles but in their Application.—From the descriptions given of the methods of fixing the horizontal and vertical positions of points when the surface of reference is a horizontal plane, it will be recognised that the *principles* of surveying are exceedingly simple. The difficulties of surveying lie not in its fundamental principles, so long as the extent of surface to be surveyed may be considered as practically plane, but in the application of them in such a manner as will best suit the character of the ground and give the *necessary degree of accuracy with economy of time and labour*. No measurement of length or angle taken on the ground can be considered as absolutely exact. There is always a certain amount of error, and similarly no measurement taken on the ground can be reproduced in absolutely exact proportion on the paper. A plan, therefore, only represents the features of the ground with a certain degree of accuracy. A very important part of the practice of surveying has to deal with the methods and precautions to be adopted in order to reduce mistakes of measurement and plotting to the smallest possible amount.

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CHAPTER II

CHAIN SURVEYING—INSTRUMENTS

CHAIN surveying is of very limited application. It is quite suitable for surveying small areas on easy ground, and might on occasion be employed for larger areas if instruments for measuring angles were lacking. It is not a practicable method for large areas or uneven ground. It has been described somewhat fully in the following pages, because much of the detail work of chain surveying is equally applicable to other methods of surveying. This chapter deals with the linear measures, British and metrical, usually employed, and with the instruments used in chain surveying—chains, steel bands, tapes, offset staff, measuring rods, arrows, ranging poles, &c., cross staff, optical square, optical prisms, line ranger, inclinometer, Abney's level, the human eye.

British Units of Linear Measurement.—The British standard for measures of length is the *Yard*. The following sub-divisions and multiples of the yard are used for convenience in stating the measurements of lengths of widely varying magnitude :—

The *Inch* = $\frac{1}{36}$ part of a yard or $\frac{1}{12}$ part of a foot. Sub-divisions of the inch are expressed either decimally or by the fractional parts $\frac{1}{2}$, $\frac{1}{4}$, $\frac{1}{8}$, &c.

The *Foot* = $\frac{1}{3}$ part of a yard. This is the unit of measurement principally used for surveying purposes in Britain and its Colonies, and in the United States of America.

The *Fathom* = 2 yards or 6 ft. This unit is often used in figuring the depths of soundings on nautical charts, and in connection with mining for stating the depths of shafts, &c.

Gunter's Chain = 22 yards or 66 ft. Used to some extent for surveying purposes in Britain, and useful for obtaining areas in acres.

Furlong = 220 yards = 10 chains.

Statute Mile = 8 furlongs = 1,760 yards = 5,280 ft.

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Metrical Units.—Throughout the principal European countries (except Russia, which has the British foot as the basis of measurement), and also South America, the unit of linear measurement is the *Metre.*

One Metre = 39·37043 ins. = 3·28087 ft.
= approximately 3 ft. $3\frac{3}{8}$ ins.

The sub-divisions of the metre are :—

Decimetre = $\frac{1}{10}$ metre.
Centimetre = $\frac{1}{100}$ metre.
Millimetre = $\frac{1}{1000}$ metre.

The multiples are :—

Decametre = 10 metres.
Hectometre = 100 metres.
Kilometre = 1000 metres.

The units most commonly used are the kilometre, metre, centimetre and millimetre.

The following approximate rules are very useful for mentally changing British units into metrical units and *vice versâ* :—

To change metres into feet divide 10 times the number of metres by 3. Thus 36 metres = $\frac{360}{3}$ = 120 ft. approximately. To change feet into metres multiply by 3 and remove decimal point one place to the left.

To change centimetres into inches multiply by 4 and remove decimal point one place to left. Thus 15 cm. = 1·5 × 4 = 6 ins. Similarly, to transfer inches into centimetres, divide 10 times the number of inches by 4. Thus 12 ins. = $\frac{120}{4}$ = 30 cm. approx.

The values determined by the above methods are wrong to the extent of fully $1\frac{1}{2}$ per cent.

Measuring Instruments.—The instruments ordinarily used in this country for measuring continuous long lengths on the ground are the "Chain" and the "Steel Band."

Chain.—Chains are made either 66 ft. or 100 ft. long. They are formed of straight links of steel wire joined to each other by three small circular or oval wire rings. The 66-ft. chain, known as Gunter's chain, is divided into 100 links each ·66 ft. long. The divisions are marked by the middle small ring at each joining. The 100-ft. chain is divided into feet. Every tenth link or foot is marked by a brass tag shaped so as to indicate its position at a

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glance. Every intermediate fifth division is also marked. See Figs. 8 and 9.

Steel Band.—The steel band consists of a thin flexible strip of tempered steel, having a cross section of about $\frac{3}{4}$ in. $\times$ $\frac{1}{40}$ in., or narrower, provided with a brass swivelling handle at each end, and marked off into feet or links. A very narrow band is less apt to break by kinking. Fig. 10 illustrates the method of marking the main divisions. The tenth, twentieth, thirtieth, and fortieth divisions from each end are marked by oval brass plates fixed on each side of the band and having one, two, three, and four round spots respectively. The centre or fiftieth division is marked by a blank oval plate on each side. The intermediate fifth divisions are marked by large-headed brass rivets, and the remaining divisions by small-headed rivets. The foot or link at each end of the band is usually divided decimally. See Fig. 11.

Etched Steel Band.—Steel bands can be obtained fully divided along the whole length into feet and inches with the divisions shown by etching. Such bands are useful where intermediate distances have to be read very exactly, but they are difficult to read and keep clean.

Steel Tape.—For exact detail measurements a steel tape is used. This consists of a light strip of steel marked as described above for the etched steel band. Usual lengths are 50 ft., 66 ft., and 100 ft. The free end of the tape has a brass finger loop, and the outside of this loop is the zero of graduation.

Linen Tape.—For the detail measurements required in ordinary surveying a linen tape is generally used. This consists of a strip of woven linen tape about $\frac{3}{4}$ in. wide, well varnished, and with the divisions and figures painted on. Feet and inches are usually shown on one side of the tape and links on the other. In many tapes the feet are marked in red figures and the inches in black figures, as shown in Fig. 13. For use in surveying, a simpler method, as shown in Fig. 12, is preferable. Linen tapes are liable to considerable extension in ordinary use and to shrinkage when wet. Tapes with interwoven metallic threads are of more constant length.

Offset Staff.—An " offset staff " consisting of a graduated wooden rod, which may be 10 ft. or 10 links long, is useful for measuring

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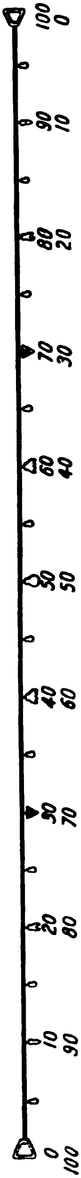

FIG. 8.—Divisions of Chain.

FIG. 9.—Detail of Chain.

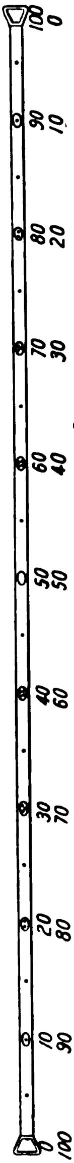

FIG. 10.—Divisions of Steel Band.

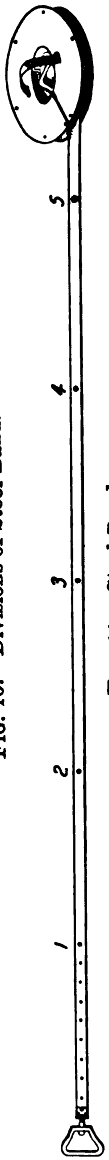

FIG. 11.—Steel Band.

FIG. 12.—Cloth Tape.

FIG. 13.—Cloth Tape.

short distances. It can be used by one person, whereas a tape requires two persons.

Wooden Measuring Rods.—Graduated wooden measuring rods, shod with metal at each end, and finished very exactly to a definite length, are sometimes used in Britain for special purposes, such as setting out steelwork. For measuring any considerable length two rods at least are used. They are placed end to end successively in the manner in which a joiner uses two 3-ft. rules to measure an exact distance. In Germany wooden rods 5 metres long are commonly used for measuring survey lines where great accuracy is required, especially in town surveying. The accuracy is said to be considerably greater than that attainable by the use of steel bands.

Metre Chain, &c.—The metre chain or band used in surveying has generally a length of 20 metres.

Arrows.—A record of the number of whole chain lengths measured is kept by means of a set of arrows, usually ten. These are formed of stiff steel wire, pointed at the lower end for fixing into the ground and having a ring at the top for carrying purposes, the overall length being usually 12 ins. The arrows are rendered conspicuous by having a piece of bright red cloth attached to the ring.

Ranging Poles.—The main survey points on the ground and, where necessary, intermediate points on the straight lines joining them are marked by means of wooden poles. These have usually a length of 6 ft., including the point, but may vary from 5 ft. to 10 ft. The poles are usually octagonal in cross-section and taper from bottom to top. They are shod at the bottom with a heavy iron point, and are painted in alternate lengths either black and white, or red and white, or black, red and white in succession. In order that the poles may, if occasion requires, be used as offset staffs, they should be painted in lengths of 1 ft. for a survey conducted with a 100-ft. chain, and in link-lengths for a survey made with a 66-ft. chain. Poles are sometimes used of round cross-section, but these are not so convenient for ranging out straight lines by the eye as octagonal poles. The gradual shading of light that occurs round a cylindrical body causes the edge of a round pole to appear very indefinite under certain views.

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Flags.—To enable a pole at a distance to be more easily picked out, a small triangular white or red flag may be attached to its top.

Laths and Whites.—Intermediate points on a line in open level ground may be very conveniently lined out with straight laths about 2 or 3 ft. long. They are also useful for marking points which have been lined out by poles, if the poles require to be removed for use elsewhere. Small straight twigs or shoots cleft at the top and holding a piece of white paper are useful for the same purpose.

Pegs, &c.—The principal survey points require to be marked in such a manner that they may be readily found at any time during the progress of the survey. In fields and soft ground stout wooden pegs are generally used. Where survey points require to be marked on hard roads, causeway, &c., pointed iron bolts, dog spikes, or large nails are used, driven in flush with the surface. In towns marks may be cut with a chisel on kerbs, pavements, &c.

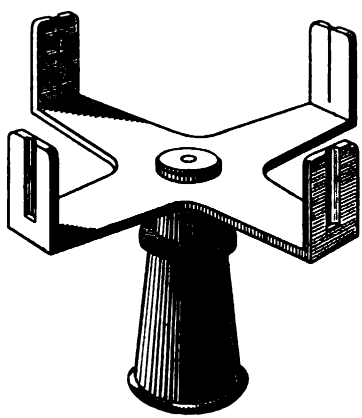

FIG. 14.—Cross Staff.

Cross Staff.—This instrument for setting off right angles, one form of which is illustrated in Fig. 14, consists essentially of a frame or a box mounted on top of a wooden rod, which is shod and pointed for fixing into the ground, the frame or box having four, sometimes eight, vertical slits arranged in its circumference, so as to give two lines of sight at right angles to each other. To set out a right angle to the chain line at a given point with this instrument, stick the rod in the ground at the point, turn one of the lines of sight till its direction coincides with the direction of the chain line as defined by ranging rods, then the other line of sight is now at right angles to the chain line, and, by viewing through the slits, a pole may be placed or a mark made at the point to which the offset should be taken. The use of the instrument to find where a perpendicular to the chain line dropped from an object, such as the corner of a

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house, will strike the chain, is not so simple, as the process is one of trial and error. To get accurate results with this instrument care must be taken to see that it is set up as nearly vertical as possible. When the vertical slits are made fairly long in proportion to the diameter of the instrument, perpendiculars may be conveniently set out on ground having a considerable slope.

Optical Square.—A more handy instrument than the cross staff and one much more used nowadays is the optical square. This is a reflecting instrument, and serves to give by means of one sight the result attained by two separate sights with the cross staff. Fig. 15 illustrates the optical theory of the instrument. SU and TU represent horizontal sections through two vertical reflecting surfaces, and ES represents the direction of a ray of light striking the mirror SU at S and making an angle C with the normal to the mirror. By the laws of reflection this ray will be reflected in a direction ST such that ST also makes an angle C with the normal. Let ST strike the mirror TU at an angle d with the normal, then the reflected ray TA will also make an angle d with the normal. It is required to find the relation existing between the angle a which the mirrors make with each other, and the angle b which the entering ray ES makes with the departing ray TA.

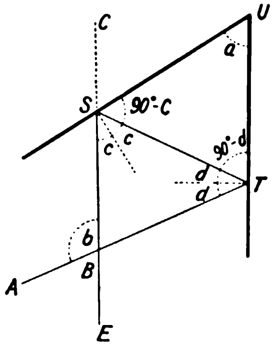

Fig. 15.—Principle of Optical Square.

In the triangle STU we have :—

$$a + (90° - c) + (90° - d) = 180°,$$

which gives : $a = c + d.$

From the triangle BST we have :—

$$b = 2c + 2d$$

$$\therefore \quad b = 2a$$

Therefore the angle between the entering and departing rays is equal to twice the angle between the mirrors, so that if the angle a be made 45° the angle b will be 90°. An eye then placed at E, and looking just over the top of the mirror SU towards a distant object C, would see in the mirror, apparently in the same vertical

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line with C, the image of an object A if so placed that the line BA is perpendicular to line BC. This is the principle of the arrange-

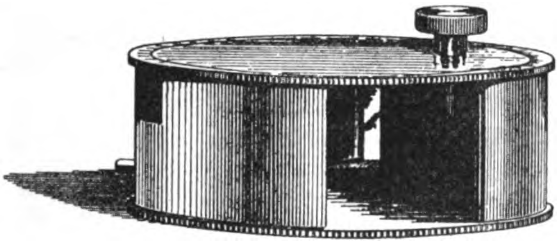

FIG. 16.—Optical Square.

ment and use of the optical square. Fig. 16 shows an external view of the instrument open, and Fig. 17 shows a plan, the lid being

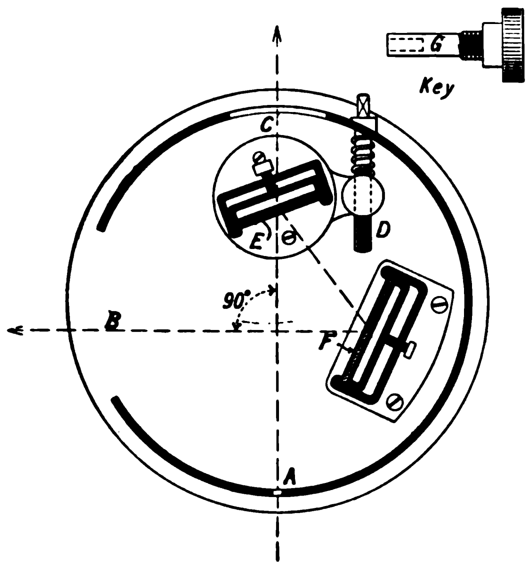

FIG. 17.—Section of Optical Square.

removed. The lower portion of the case contains the two mirrors and has a vertical rim in which are formed the eyehole A, the aperture C for viewing the forward object, and the hole B for viewing the side object. The lid or top portion of the case is similar to the lower portion, over which it slips. When the instrument is

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not in use all the openings are closed by rotating the lid. In Fig. 17 the mirror F is firmly fixed to the bottom of the case. The mirror E is attached to the circular base shown, and this is connected to the case by a centre pin, about which it can rotate. The rotation of the mirror, required for purposes of adjustment, is effected by means of the screw D working in an arm attached to the base of the mirror as shown. The adjustment of the verticality of the mirror is effected by means of two screws whose heads project through the bottom of the case. A key G, which screws into the lid of the box when not in use, is provided for the adjustment of the mirror E. The instrument complete is about 2¼ ins. diameter and ⅞ in. thick.

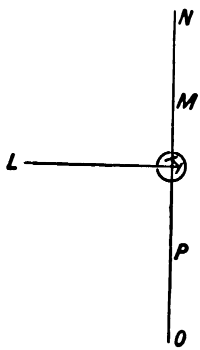

Fig. 18.—Use of Optical Square.

Use of Optical Square.—Fig. 18 illustrates the use of the optical square. Let a straight line ON be defined by poles erected at points O, P, M and N. It is required to find at what point on this line a perpendicular from a point L will fall. Holding the instrument to the eye and looking directly towards the pole at M walk forward along the straight line until the doubly reflected image of the pole or object at L is seen immediately under the pole at M. The optical square is then at the point required. The instrument is also used for setting out a perpendicular to a straight line from a given point. The surveyor in this case stands at the given point and sends an assistant with a pole to the required distance in a direction as nearly at right angles as can be estimated. He then directs his assistant backwards or forwards until the reflected image of his pole appears in the same vertical line with the forward pole. When the perpendicular has to be set out to the right hand the instrument must be held in the left hand. To set out a perpendicular to the left hand the instrument must be held other side up in the right hand. It is better to use the same eye in all cases, say the right eye. The surveyor should experiment for himself with a plumb-bob to find out in what position he must stand so

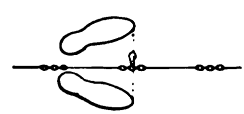
Fig. 19.—Stand for Use of Optical Square.

that the optical square may be held directly above the required point on the chain line. Fig. 19 shows the position the surveyor should take up in order to set off a right angle at the 50 mark on the chain, using the instrument at the right eye, the point on the chain being in line with the toes.

Use of Optical Square on Sloping Ground.—The general usefulness of the optical square depends on the fact, not always recognised, that it can be used for accurately setting out a right angle on ground of any steepness provided it is possible to obtain a horizontal line of sight in the other direction. This use of the optical square is illustrated in Fig. 20. The surveyor standing in the field at A desires to fix a pole at C on the railway embankment in a line at right angles to the chain line defined by the poles at B and D.

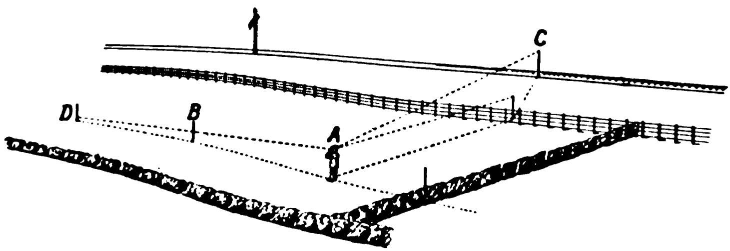

FIG. 20.—Setting out a Right Angle on Sloping Ground.

Let the horizontal line of sight from the eye at A strike the first pole at point B. Then the line AC and its horizontal projection on the ground will be truly at right angles to the line ABD, when on tilting the instrument so as to bring the image of the vertical pole at C into view simultaneously with the pole at B, *any point* of the pole C can be brought into coincidence with the point B. The angle BAC is then a right angle set out in a sloping plane, the side BA being, however, horizontal, and the projection of AC on the ground is also at right angles to AB. This may be shown by placing a right-angle set-square in a sloping position with one base resting on a horizontal table.

Testing the Optical Square.—The best method of procedure in testing and correcting the optical square is as follows: On fairly uniform ground set out three poles A, B, C, in a straight line at, say, 50 yards apart, as shown in Fig. 21. With the instrument at

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B sight towards C and set out the angle CBd_1, supposed to be a right angle, and fix a pole at d_1. Then turn round and, with the instrument turned upside down, sight towards A, set out the same angle ABd_2, and fix a pole at d_2, the distance Bd_2 being equal to Bd_1 and, say, about 50 yards. If d_2 coincides with d_1 the instrument is in correct adjustment, the angle set out each time being 90°. If, however, d_2 does not coincide with d_1, fix a pole at D midway between the two points. Then the angle CBD will be a right angle, and with the instrument at B the key must be used to turn the movable mirror until the image of the pole at D is seen to coincide with the pole at C. Then test further by sighting towards A. If the image of D now also coincides with the pole at A the instrument is in correct adjustment.

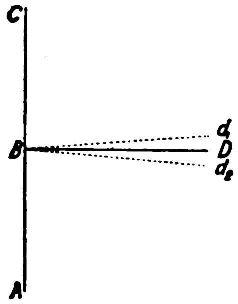

FIG. 21.—Testing Optical Square.

Prism Square.—The most reliable hand instrument for setting out right-angle offsets, &c., is the prism square. The principle of one form of the instrument is shown in Fig. 22. The prism consists of a small block of glass, the upper and lower parallel surfaces having the form of right-angled isosceles triangles. All the faces

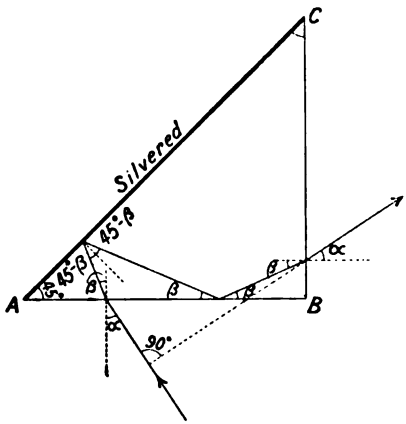

FIG. 22.—Principle of Prism Square.

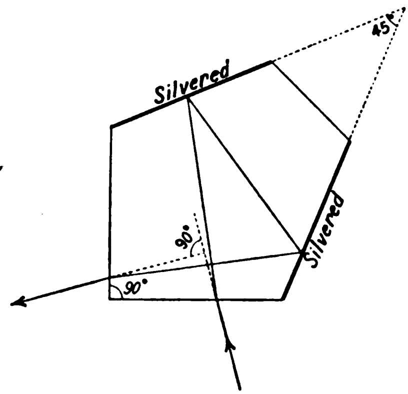

FIG. 23.—Principle of Five-sided Prism.

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are ground and polished to make the exact required angles with each other, and the hypotenuse edge is silvered. A ray of light entering side AB in an oblique direction near the acute corner A, as shown in the diagram, and making an angle α with the normal to the face, is refracted and enters the glass, making an angle β with the normal such that $\sin \alpha = U \sin \beta$ where U is a coefficient depending on the kind of glass. The ray then follows the path shown on Fig. 22, and a study of the diagram shows that the emerging ray is at right angles to the entering ray.

The instrument is used in a similar manner to the optical square. A forward object is sighted directly over or under the prism, and simultaneously the image of a side object is seen in the prism after two refractions and two reflections. The right angle is given when the image of the side object appears in coincidence with the forward object.

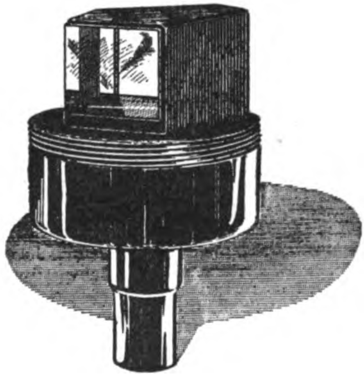

Fig. 24.—Zeiss Prism Square.

Fig. 23 shows the principle of another form of the prism, and Fig. 24 shows a view of the corresponding instrument complete. As compared with the usual reflecting form of optical square, the prism has the advantage of greater clearness. It can be used almost as long as objects are distinctly visible, while the optical square fails long before dusk. The prism has the further great advantage that, if it is correct to begin with, it remains correct so long as it is at all serviceable.

Wooden Set-square.—For taking accurate small offsets, as required in town surveying, a large triangular wooden set-square is sometimes used. Such a set-square, with the perpendicular sides, say, 6 ft. and 4 ft. long, is also useful in setting out small details of buildings, bridges and similar work.

Line Ranger.—This is a hand instrument used for obtaining with a close degree of approximation an intermediate point on the line between two survey stations without sighting from one end of the

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line. The principle of a reflecting form of the instrument is shown in Fig. 25. Two small mirrors are fixed exactly at right angles to one another, one on top of the other, and mounted in a case, or simply on a handle. An examination of the diagram shows that if an object A is seen in the upper of the two mirrors immediately over their crossing point, then an object B will be seen in the lower mirror directly under the image of A, provided B, the instrument

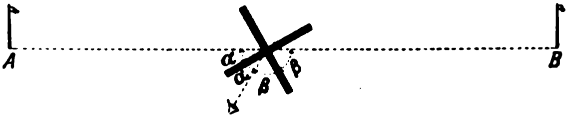

FIG. 25.—Principle of the Line Ranger.

and A are in one straight line. In using the instrument, therefore, the observer, having placed himself approximately in line, holds the instrument in front of his eye, and turns it so as to bring the image of A to about the centre of the upper mirror. Looking in the lower mirror he will see the image of B, but, unless by good luck, it will not appear directly under the image of A. He then moves backwards or forwards at right angles to the direction AB until the images of A and B appear exactly in the same vertical

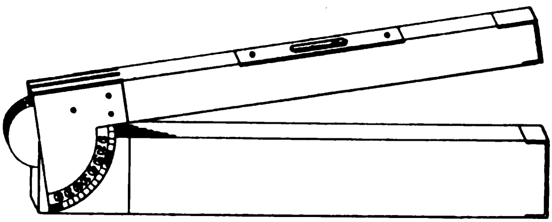

FIG. 26.—Inclinometer.

line. The instrument is then in line between A and B and its position may be marked by a pole. Another type of the instrument consists of two prisms of the form shown in Fig. 22, mounted one on top of the other with their long sides at right angles to each other. It is used in the same manner as the reflecting type.

Inclinometer.—In order to determine the true horizontal distance between two points on sloping ground when the measurement has been made on the slope, it is necessary to know the vertical

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angle of inclination of the ground. A simple instrument suited for this purpose is shown in Fig. 26. In using the instrument as illustrated in Fig. 27, a pole or staff is laid on the ground, and on it the lower bar of the instrument is placed. The upper bar is then tilted up to the horizontal position, as shown by the bubble of the spirit level coming to the centre of its run. The inclination of the ground is then read off on the graduated arc to about the nearest half degree. The instrument may be used in a similar manner for rapidly taking rough cross-sections on steep ground.

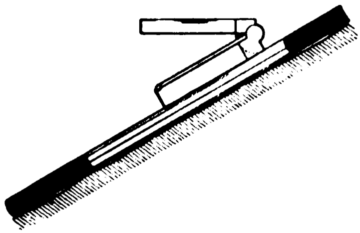

Fig. 27.—Use of Inclinometer.

Abney's Level.—Abney's level is a useful hand instrument for measuring vertical angles, and is based on a very neat device. The principle of the instrument is shown in Fig. 28. A square

Fig. 29.

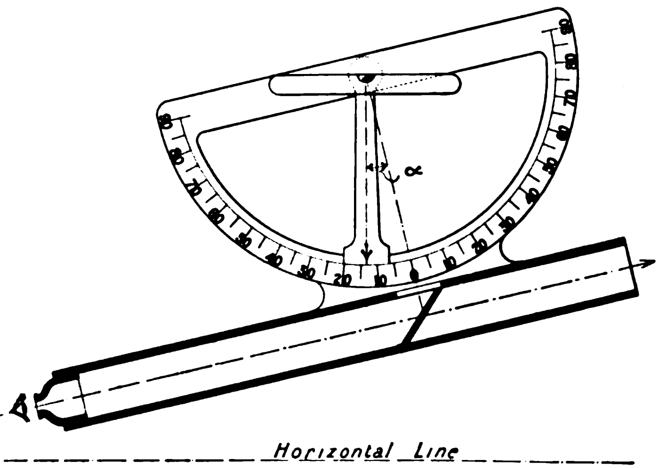

Fig. 28.—Principle of Abney's Level.

tube about 4½ ins. long has a small eye-hole at one end. Near the other end a mirror inclined at 45° to the axis of the tube extends across half its width, the other half of the tube being open. A horizontal scratch extends across the centre of the mirror and the same line is produced across the open half of the tube by a fine

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hair or wire. Attached to the tube is a graduated arc of a circle about 3 ins. diameter, whose centre lies on a line drawn from the centre of the mirror at right angles to the axis of the tube. An index arm is hinged to the centre of the arc. A bubble tube is attached at right angles to the index arm in such a position that when the bubble is at the centre of its run it is also at the centre of the arc. An opening in the top of the tube allows the bubble to be seen in the mirror. An inspection of the diagram shows that when the bubble is seen bisected by the scratch in the mirror, it is at the centre of its run and the index arm is vertical. The angle a which the index reads on the arc is then equal to the angle a which the line of sight makes with the horizontal. Fig. 29 shows the field of view in the tube when a sight is taken to the top of a pole. The hatched portion indicates the mirror with the image of the bubble bisected by the scratch. The unhatched portion shows the view through the open half of the tube.

In using Abney's level to find the inclination of a line on the ground it is necessary to sight to an object at the same height above the ground as the observer's eye. The top of a pole of proper height forms a convenient object, but it will often be found expedient to observe simply to the head of an assistant.

The Human Eye.—The human eye is an optical instrument which every surveyor on occasion uses instead of level, plumb-line or square. Unless where extreme accuracy is required, he may use it to tell him when the slope of the ground is such that its effect may be neglected in measuring along a survey line. In surveying details and taking offsets he uses it continually to tell him when the tape is held horizontally. He may use it to tell him whether a ranging pole or level staff is standing vertically, or whether one direction is at right angles to another. On fairly level and uniform ground, or where there are known horizontal and vertical lines to refer to, the normal eye will in the above cases give good approximate results. On the other hand, in presence of certain configurations of ground, and under certain influences, it will give indications which are not only unreliable, but are absolutely misleading. A beginner, levelling on steep ground and working up or downhill, soon finds that in setting up his instrument he may easily misjudge the level of the bottom or top of the staff by several feet at quite

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a short distance away. If he has much levelling to do along roads in undulating country he may happen on a stretch where his eye tells him definitely he is going downhill while his level proves he is going uphill. When walking along a straight railway embankment on steep sidelong ground he may notice that the permanent way appears canted up towards the deep side of the embankment while as a matter of fact the rails are level across. With no known vertical or horizontal object to guide him let him erect a ranging pole on steeply sloping ground, so as to be as nearly vertical as he can judge by the eye. He will find on testing with the plumb-bob that the pole is hanging considerably downhill. In the above-mentioned cases the configuration of the ground acts to produce an optical illusion. Due to a different influence, centrifugal force, is the familiar illusion whereby to a passenger in a train travelling round a curve external objects appear off the perpendicular. Enough has perhaps been said to show that the eye is not an independent instrument and, therefore, should not be used for estimating verticals, horizontals, &c., where accuracy is required.

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CHAPTER III

CHAIN SURVEYING—FIELD OPERATIONS

In this chapter consideration is given to the methods of ranging out straight lines by the eye on level ground, and over rises and hollows, as would be required in setting out the base-lines for a chain survey. The measurement of a distance with the chain is also described in detail, and attention is drawn to the more frequent sources of error in reading the chain. The methods of obtaining the true horizontal distance on sloping ground are fully gone into.

Survey Party and Equipment.—The surveyor will have two assistants, who will handle the chain and other instruments, while the surveyor himself directs the operations and records the measurements in the survey-book. The party will be equipped with the following instruments:—

Chain or steel band (66 ft. or 100 ft.).
50 ft. or 66 ft. linen tape.
Set of ten arrows.
Set of ranging poles, say six.
Optical square or prism.
Inclinometer.
Plumb-bob.

Ranging Lines.—The end points or stations of the survey line having been fixed, it is necessary, if the line is of considerable length, to establish and mark by poles, laths, or otherwise, some intermediate points for the guidance of the chainmen. Suppose, in the first instance, that the stations are fully 1,000 ft. apart, with fairly uniform and unobstructed ground between, and that it is desired to mark three intermediate points. Ranging rods are first set up at the end points and their verticality assured by testing with the plumb-bob. The surveyor remains at one station and directs an assistant to take with him the number of poles required for the

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intermediate points and to proceed first to the most remote point. He will there, having guided himself as nearly into line as he can judge, stand facing the surveyor, and hold a pole erect and out to one side of himself so that the surveyor may have a clear view of it. He will then move in the direction signalled to him by the surveyor, a motion of the surveyor's right hand meaning that he is to move laterally towards the surveyor's right hand. A good method when the assistant is standing wide of the line is for the surveyor to keep his arm held out in the direction in which the assistant has to move. The latter will then keep moving in that direction until he sees the arm drop, when he will know he is nearly in line. He will then hold the pole erect, point on ground, and move it a little at a time according to the signals until it is in correct position. The signal for this is usually given by the surveyor raising both arms and bringing them rapidly down together, the gesture indicating that the pole is to be "planted" or stuck into the ground. On receiving this signal the assistant will press his pole a little distance into the ground, plumb it carefully, and then step aside to let the surveyor get another view. He will then shift it a little or press it home, according to the signal he receives. He must take particular care to leave it perfectly plumb. He will then proceed to the second point, and having now two poles behind him, he should turn round and rapidly put his pole into line with these two. Then, on getting the signal from the surveyor, he will take a very small shift and rapidly arrive at the correct position. He will proceed in the same manner for the other points.

The assistant who has to set poles in a survey line should pay attention to the following points: Before setting out to give the points he should, whenever possible, take a view along the line from one end station and take note of any mark, such as a tuft of grass, weed, stone, &c., occurring near the line at the position of his first point. He will thereby be enabled to hold the pole nearly on line at the first attempt, thus saving time. When carrying several poles with him, he must be very careful not to hold a pole out for lining with one hand while at the same time retaining the spare one or two in the other hand, and in view of the surveyor, as much confusion and annoyance may thereby be caused. When he is holding out a pole for lining, the spare ones should be lying on the ground.

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It cannot be too much emphasised that good lining out can only be accomplished when the poles are held and planted truly vertical. The assistant must, therefore, train himself to hold the poles correctly and to test them rapidly. Rough-and-ready approximations to the vertical may be got, in the absence of a plumb-bob, by holding a pole lightly at the top between the thumb and a finger, and letting it hang free, or by dropping a small stone. It is better, however, to improvise a plumb-bob by attaching any small heavy object to the end of a string.

The usual method adopted by the person who is directing the poles into line is as follows: Standing or crouching behind the station pole he puts one eye in line with the outside edges on one side of the two station poles and signals the assistant with the third pole until its edge appears in the same line. He tests by looking along the edges on the other side. To get good results he must not sight from close beside the station pole, but from some distance behind it. The following is a more rapid and satisfactory method than the above: The surveyor stands with both eyes open a few yards behind the station pole and looks fixedly at the distant pole. He sees two images of the near pole, as it is out of focus. He places himself so that the distant pole appears central between these two images and then directs the intermediate pole into position, so as to completely cover the distant pole. It is then in correct line.

In practice, survey lines may often be ranged by the method of production. A pole is fixed at one end of the line and another is placed a short distance ahead, say, 200 or 300 ft., in the direction in which the line is to run. A third pole is then fixed a similar distance ahead by sighting back on the two already planted, and so placing it that all three appear in line. The line is extended to the required length by fixing additional poles successively in the same manner. On uniform ground straight lines 2,000 ft. long may be run very accurately by this method.

When the purpose of ranging a straight line is merely to enable the distance between the end stations to be measured, little refinement is required in fixing the intermediate points. A deviation of 1 ft. in a line several hundred feet long would give rise to no appreciable error in the length of the line. If offsets were taken from the line, however, they would be in error to the extent of the

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deviation. Particular care must, therefore, be exercised in the ranging of lines from which offsets are to be taken.

Ranging Line across a Hollow.—When a depression intervenes between the end stations, so that poles held at intermediate points come wholly below the direct line of sight between the stations, or when a hill occurs to prevent the one station from being visible from the other, the ranging of an accurate straight line becomes a difficult matter. On ground which gives rise to many such cases good results cannot be expected from chain surveying. The usual method of ranging out a line across a hollow between two points A and B (Fig. 30) is as follows: The surveyor standing at A directs an assistant to a point C, such that the top of a pole held there is at the level of the line of sight from A to the bottom of pole at B.

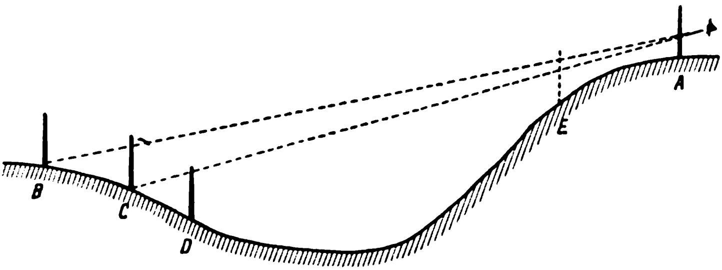

Fig. 30.—Ranging Line across a Hollow.

He then signals the pole at C into line, which must be planted perfectly vertical, as otherwise the bottom might be out of line. A pole may then be lined out to a point D further downhill, by sighting to the bottom of the pole at C. Successive points may be put in line in this manner until perhaps a point in the hollow is reached, which is invisible from A.

A more expeditious method of proceeding after the pole at C is fixed is for the assistant there, sighting to the bottom of pole A, to line out a pole held by the surveyor at E. The surveyor at E will then line out his assistant at D, and so on, each moving downhill alternately.

A method which is often useful, and is generally appropriate when the valley to be crossed is fairly deep, is to hang a plumb-bob over the station A (Fig. 30) by the method shown in Fig. 31. The surveyor then stands as far back from the plumb-line as he can

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without limiting his view into the valley, puts his eye into range with the pole at B and the plumb-line, and then sights down the plumb-line and signals the pole at C or D into line. He must practically sight to B and C simultaneously.

Ranging Line over a Hill.—The method of ranging a straight line between two points A and B over an intervening hill is illustrated in Figs. 32 and 33. The surveyor goes to a point C on one side of the crown of the hill, where he can just see the top of the pole at B, and as nearly in the straight line between A and B as he can estimate. He directs his assistant to go down the other side of the hill until he can just see the top of the pole at A, and then to put his pole into range with A and C. The surveyor standing at C now sees that the assistant's pole at D is not in line between poles C and B. He, therefore, directs the assistant to move laterally to point D′

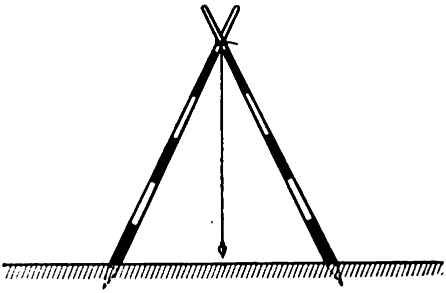
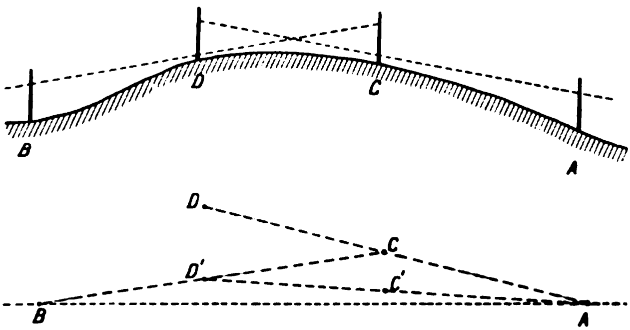


FIG. 31.—Device in Ranging Line.



FIGS. 32 and 33.—Ranging Line over a Hill.

in line between C and B. The assistant at D′ now directs the surveyor to C′ in line between D′ and A. They proceed in this manner, each putting the other in line alternately, until it is found that the intermediate poles range simultaneously with the extreme poles. It need hardly be pointed out that the above procedure, which

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necessarily involves sighting to the upper ends of poles, is not conducive to accurate work, and that, if more than one undulation occurs between the stations, it may be quite impracticable to run the straight line without the use of the theodolite.

Ranging past Obstacles.—Objects such as trees, hedges, &c., occurring on a survey line may often form as great hindrances to the ranging of a line as hills or valleys. Some devices for ranging past or over obstacles are described in Chapter VII. As no two cases happen exactly alike, the surveyor must hold himself ready to invent or adapt a device to suit each circumstance.

Chaining Survey Lines.—A new steel band should, before using, be tested on an official standard at ordinary temperature and its error, if any, noted. It will not require further testing unless after repairing, due to breakage, or if the handle swivels should become worn. The official standard, as laid down in some of the large towns, for testing 100-ft. chains is illustrated in Fig. 34. At one end of a heavy, continuous horizontal granite base a block of brass with a vertical face is fixed. This face is the zero of the

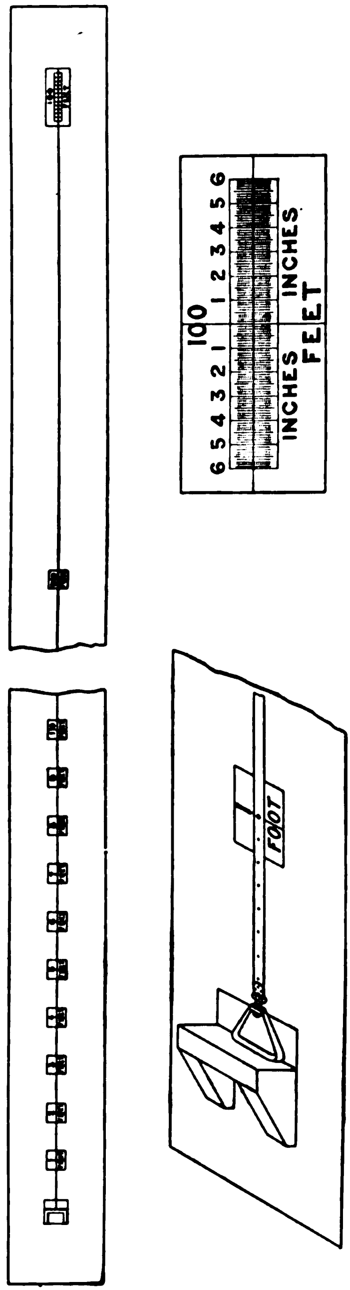

Fig. 34.—Official Standard for Testing Chain.

standard, and one handle of a chain is held against it in testing. At every 10 ft. along the base a brass plate is sunk in flush with the granite with a score across, marking the distance. At the 100-ft. end a brass plate contains the score representing the true length of 100 ft., and is further divided into inches and decimals for a short distance on each side of the 100 mark to enable the error of the chain under test to be ascertained.

When the instrument to be used is the chain, its length must be constantly checked. In ordinary use the joint rings and hooks of the links are liable to open and stretch, thus increasing its length, while it may be shortened by links getting bent, or the rings getting clogged with dirt or grass, and an error of 2 or 3 ins. in its length

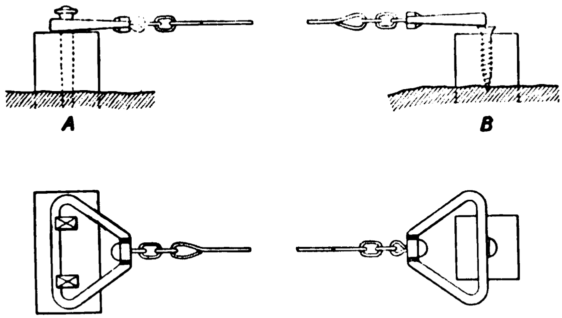

FIG. 35.—Field Standard for Testing Chain.

may readily creep in. Where the chain is in constant use a pocket steel tape should be kept and compared with it every day. When the chain is too long a ring or two may be taken out. When too short, rings may be flattened, thus making them longer; and when necessary, additional rings may be inserted. Where several rings require to be taken out or inserted, they should be distributed at intervals over the length of the chain, in order to avoid large error at any intermediate point. In all cases before testing a chain it should be carefully examined and cleaned, all bent links should be straightened, and kinks at the joints, if any, undone.

Field Standard.—A convenient standard for testing a chain may be set up at the site of a survey by means of two stout pegs, A and B, driven into the ground at the proper distance apart and with nails fixed in their heads, as shown in Fig. 35. One handle of the chain

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can be hooked on to the pair of nails on A, and when the chain is stretched out its other handle should be over the nail on peg B. With a standard arranged thus the chain can be tested and corrected by one person.

Chaining on Level Ground.—The surveyor directs the operations and books the measurements. His two assistants handle the chain. The one at the forward end of the chain is called the "leader," the other is the "follower." It is assumed that the distance between the end stations of a survey line already ranged out is about to be taken, no intermediate measurements or offsets being required. The leader receives a full set of arrows and must count them to see that he has the correct number, viz., ten. If the leader is new to the work, the surveyor must instruct him as to the correct method of holding the chain handle, working the chain into line, and fixing the arrow.

The link chain is generally kept gathered up into a sheaf and tied with a strap passing round its middle and through the two handles. To undo the chain, having taken off the strap, the chainman takes both handles in the left hand, allows some links next the handles to come loose, then takes the rest of the bundle in his right hand and throws it away from him. The chain will unfold itself and lie along the ground double. If the two halves are entangled, an assistant should take hold of the 50 tally, pull the chain taut and untwist till the two halves are clear of each other. The chain may now be straightened out to its full length. Before using it, a careful inspection should be made to see that it is not kinked at any of the joints, and that none of the links are bent. The steel band is usually contained in a case and is undone by one assistant holding the case while the other pulls out the band.

Before laying out the first chain length the leader should look along the line from the commencing station and note his direction, so that when he has drawn the chain out to full stretch he may not be far from the correct line. As the leader advances, the follower holds his end of the chain in his hand and imposes a gentle pull, carefully avoiding any jerk, when the leader has reached the limit of the first length. The leader stops, turns round and faces the follower, and, crouching down, holds the handle of the chain in his right hand, with an arrow held vertically against its end.

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The point of the arrow should project an inch or two under the handle, and both should be held together out to the side so that no part of the leader's body may prevent the follower from seeing the arrow or the next forward pole. The follower meanwhile holds his end of the chain to the centre of the station mark, looks towards the forward pole, and by word or signal directs the leader to move the chain and arrow in one direction or the other, so as to bring them into line. The leader moves the chain to the side, and straightens it out by sending a wave along so as to lift it off the ground gradually from end to end, while simultaneously giving it a slight flick to the side and keeping a small amount of tension on it. It requires some experience to do this without transmitting an awkward jerk to the follower.

When the 100-ft. chain is being used the following procedure may be preferable. The chain having been straightened out but not brought into correct line, the follower directs the leader to move his handle and arrow laterally until the arrow is in line. It will be short of its proper distance. The leader puts the point of the arrow into the ground, holds it there with his left hand and with his right hand straightens out the chain past the arrow and then moves the arrow forward to the end of the chain. On receiving the signal "Mark," or "Right," or "Down," from the follower, the leader presses the arrow firmly and vertically into the ground. The leader and follower then proceed, each holding an end of the chain. If there are two or more poles ranged out ahead, the leader should keep himself in line by their aid, and should count his paces so as to be ready to stop at once on feeling a gentle pull from the follower. When there is no pole left at the commencing station the leader should look back along the first chain after it has been lined in, and note some object in range therewith which may serve as a "back mark" in lining succeeding chain lengths.

The follower has a record of the number of whole chain lengths measured in the number of arrows which he has picked up. When ten chain lengths have been measured and the leader has put in his last arrow he leaves the chain lying on the ground and waits until the follower comes up. The follower picks up the last arrow, taking care to mark the point by some other means, counts the arrows to see that he has ten, then hands them back to the leader.

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The surveyor notes in his book the distance 1,000 (feet or links), the leader again pulls the chain ahead, and chaining proceeds as before, until the leader fixes an arrow within a chain length of the end station. Suppose that the total distance is 1,457 ft. The leader, having put in his fourth arrow, pulls the chain *past* the station until the follower reaches the arrow. The follower holds his handle to the arrow while the leader, having come back to the station, pulls the chain taut and reads the distance. The surveyor checks this, finds that the follower has four arrows, and makes sure that this tallies with the number retained by the leader, and sees from his book that ten complete chains have been already measured. The total distance is, therefore, 1,457 ft., which he enters in his book.

The most common error in reading an intermediate distance on the chain arises from the fact that each division represents two different distances, according as the measurement is taken from one end or the other. Unless care is taken to read from the proper end an intermediate distance, such as 48 ft., may quite readily be read and booked as 52 ft., and while it is not so common to make the mistake of reading 71 when the distance should be 29, it is quite common to read 31 instead of 29. Mistakes are most readily made in reading points near the centre of the chain, and the resulting error, being comparatively small, is sometimes very difficult to locate.

Precautions to be observed by the Leader and Follower.—To recapitulate shortly, the leader should observe the following precautions in chaining:—

He should count his paces and be ready to stop when each chain length is out, and should stop at once on feeling a slight pull.

He should guide himself almost into line.

He should hold the arrow vertical and fix it vertical.

He should carefully avoid tugging or pulling violently on the chain in getting it into line.

The following precautions should be observed by the follower:—

He should stop the leader at the right moment by a gentle pull.

He should never allow his handle to drag on the ground,

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nor pull up the leader with a jerk by putting his foot on it.

In guiding the leader into line when there is only one mark ahead to line to he should be very careful to sight from a point directly above the arrow. This is important and difficult.

Chaining on Sloping Ground.—If the line joining two points whose distance apart is to be determined is not level, the distance measured directly along the surface of the ground will not be the true horizontal distance between the points, but will be somewhat greater. Any distance so measured must, therefore, be corrected to allow for the slope, or some other method of measuring must be

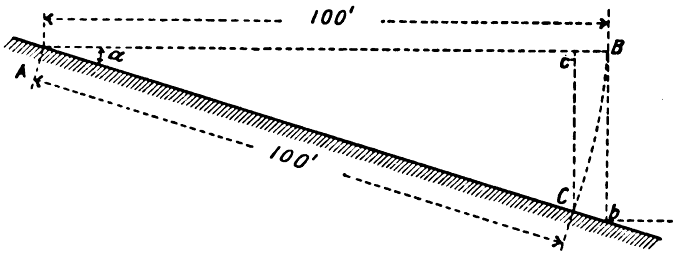

FIG. 36.—Measuring on Sloping Ground.

used which will get rid of the effect of the slope. Three methods of measuring on sloping ground are illustrated in Fig. 36.

Method of Stepping.—The first method is known as "stepping," and is best accomplished when chaining downhill. The follower holds his end of the chain firmly at the starting point A, while the leader stretches the chain, or a portion of it, out horizontally in the air in correct line as directed by the follower, and pulls it tight. The end of his handle comes to B, and using a plumb-bob he transfers this point vertically to the ground to point *b* and marks it with an arrow. The chain is then pulled forward, the follower holds his end to point *b*, and the same process is repeated. For accurate work the length of each step should not amount to 50 ft. when a heavy chain is used. Much better results can be got by lining out the chain full length down the slope, leaving it lying on the ground as a guide to the direction, and using a light steel tape for the

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stepping. This can be stretched nearly horizontal in a length of 50 ft. The chief source of inaccuracy in stepping is due to the tape or chain not being held horizontal. Both the leader and the follower are badly placed for estimating horizontal direction by the eye, and on steep ground even the surveyor standing apart may be deceived. The only reliable method is for the leader to raise or lower the tape until it makes a right angle with the plumb-line as nearly as can be judged.

In stepping down a slope in short lengths it is desirable that the leader should mark the intermediate points between the ends of each chain by means of twigs or in some other temporary manner. If he uses arrows for the intermediate points, he must be careful to get them back from the follower at the end of each whole chain length, otherwise a miscount of the total number of chain lengths will almost surely result.

Method by Measuring on Slope and applying Correction to Total Length.—The second method consists in measuring the total length between the points along the sloping surface of the ground, and thereafter calculating the deduction which must be made to reduce this length to the true horizontal distance. In Fig. 36, AC represents a chain length measured down the slope, and Ac is the corresponding true horizontal distance, while cB represents the amount to be deducted from AC to arrive at the true horizontal distance.

Let $\alpha =$ angle of slope.
Then $Ac = AC \cos \alpha$.
and $cB = AC - Ac = AC\ (1 - \cos \alpha)$.

If, therefore, a total length L has been measured along a slope of angle α, the true horizontal length will be $L \cos \alpha$ or the length measured will be too great by the amount $L\ (1 - \cos \alpha)$. If the length between two points consists of several stretches with different slopes, a separate calculation must be made for each stretch. This method of measuring on sloping ground is only convenient where there are no intermediate points whose distances require to be recorded.

The table on p. 38 gives the true horizontal length corresponding to unit distance measured on slopes up to 30°.

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Table of Horizontal Lengths corresponding to Unit Lengths measured on Slope.

Slope in Degrees.	Horizontal Length.	Slope in Degrees.	Horizontal Length.	Slope in Degrees.	Horizontal Length.
1	·99985	11	·98163	21	·93358
2	·99939	12	·97815	22	·92718
3	·99863	13	·97437	23	·92050
4	·99756	14	·97030	24	·91355
5	·99619	15	·96593	25	·90631
6	·99452	16	·96126	26	·89879
7	·99255	17	·95630	27	·89101
8	·99027	18	·95106	28	·88295
9	·98769	19	·94552	29	·87462
10	·98481	20	·93969	30	·86603

Example.—In measuring a total distance of 667 ft. along the ground between two points, the first 200 ft. were on a slope of 5°, the next 300 ft. on a slope of 8°, and the remaining 167 ft. on a slope of 6°. Using the above table, find the true horizontal distance between the points.

For 5° slope, ·99619 × 200 = 199·238
,, 8° ,, ·99027 × 300 = 297·081
,, 6° ,, ·99452 × 167 = 166·085

Horizontal distance = 662·404

Method by Measuring on Slope and applying Correction on Ground at each Chain.—In this method the chain is laid out down the slope, and an arrow is put in temporarily at C (Fig. 36). The slope angle having been measured with the inclinometer, the additional distance C*b* required to make A*b* have a true horizontal length of 100 ft. or links can be calculated or found from a table

$$
\begin{aligned}
\mathrm{A}b &= \mathrm{AB} \sec \alpha = \mathrm{AC} \sec \alpha \\
\therefore \mathrm{C}b &= \mathrm{AC} \sec \alpha - \mathrm{AC} \\
&= \mathrm{AC} (\sec \alpha - 1).
\end{aligned}
$$

The distance C*b* having been found, it is measured off on the ground by means of tape or foot-rule, and the arrow is advanced

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to *b*. Each chain length is similarly corrected, care being taken to determine the slope anew at each change of inclination. This method is generally more convenient than stepping where the slopes are moderate and the ground fairly uniform. The slope angle is determined closely enough by an inclinometer of the hinged type placed on a pole or offset staff laid on the ground.

The following table shows the corrections required per chain length of 100 ft. or 66 ft. for slopes up to 20° :—

TABLE OF CORRECTIONS TO BE MADE ON THE GROUND FOR CHAIN LENGTHS MEASURED ON SLOPE.

Slope Angle. Degrees.	Correction to be made on ground, measured on the slope.				
	Feet per 100 ft. or links per 100 links.	Per 100 ft.		Per 66 ft.	
		Ft.	Ins.	Ft.	Ins.
1 . . .	·01523	0	$0\frac{3}{16}$	0	$0\frac{1}{8}$
2 . . .	·06095	0	$0\frac{3}{4}$	0	$0\frac{1}{2}$
3 . . .	·13723	0	$1\frac{5}{8}$	0	$1\frac{1}{8}$
4 . . .	·24419	0	$2\frac{15}{16}$	0	$1\frac{15}{16}$
5 . . .	·38198	0	$4\frac{9}{16}$	0	3
6 . . .	·55083	0	$6\frac{5}{8}$	0	$4\frac{3}{8}$
7 . . .	·75098	0	9	0	$5\frac{15}{16}$
8 . . .	·98276	0	$11\frac{13}{16}$	0	$7\frac{3}{4}$
9 . . .	1·24651	1	$2\frac{15}{16}$	0	$9\frac{7}{8}$
10 . . .	1·54266	1	$6\frac{1}{2}$	1	$0\frac{1}{4}$
11 . . .	1·87167	1	$10\frac{7}{16}$	1	$2\frac{13}{16}$
12 . . .	2·23406	2	$2\frac{13}{16}$	1	$5\frac{11}{16}$
13 . . .	2·63041	2	$7\frac{9}{16}$	1	$8\frac{13}{16}$
14 . . .	3·06136	3	$0\frac{3}{4}$	2	$0\frac{1}{4}$
15 . . .	3·52762	3	$6\frac{3}{8}$	2	$3\frac{15}{16}$
16 . . .	4·02994	4	$0\frac{3}{8}$	2	$7\frac{15}{16}$
17 . .	4·56918	4	$6\frac{13}{16}$	3	$0\frac{3}{16}$
18 . . .	5·14622	5	$1\frac{3}{4}$	3	$4\frac{3}{4}$
19 . . .	5·76207	5	$9\frac{1}{8}$	3	$9\frac{5}{8}$
20 . . .	6·41778	6	5	4	$2\frac{13}{16}$

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CHAPTER IV

CHAIN SURVEYING—RUNNING A SURVEY LINE

In this chapter consideration is given to the methods of fixing points in relation to a survey line by means of offsets and ties. The measurements required for properly fixing buildings, irregular boundaries, &c., are then dealt with, and the running of a survey line is described in detail. Methods of booking the work are described and examples are given, illustrating the booking of survey lines.

Fixing Positions of Objects relative to Points on a Survey Line.—The determination of the positions of objects in relation to points fixed by measurement along a survey line usually constitutes the bulk of the detail work in surveying. The methods adopted for a survey line in a chain survey apply equally to a survey line in a triangulation or traverse survey. Two methods are in common use. In the first method, that of "offsets," the point is located by the measurement of a distance and an angle (usually 90°) from a point on the chain line.

In the second method, known as the method of "ties," the distance to the object is measured from two separate points on the chain line, the three points forming as nearly as possible an equilateral triangle.

The two methods applied to fixing the corners of a building are illustrated in Figs. 37 and 38. In Fig. 37 the corners A and B of a building are fixed by perpendicular offsets of 30 ft. 0 ins. and 20 ft. 0 ins. from points at distances 530¾ and 564½ respectively along the chain line. If the building were rectangular, the only further measurement required to enable its position to be plotted on paper would be its width. The surveyor would, however, have no check on the accuracy of the work plotted from these measurements, and a mistake made either in the field measurements or in the plotting would pass unnoticed. By measuring and recording

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the length AB a check on the accuracy of the plotted points A and B would be obtained, and a check on the directions of the sides AD and BC would be got by noting the points where these directions cut the chain line. In general, to obtain satisfactory results, in work which has to be plotted to a fairly large scale, all the measurements indicated in Fig. 37 should be recorded.

In Fig. 38 the method of fixing the building by means of ties is illustrated. Point E is located by the lengths 31 ft. 6 ins. and 33 ft. 3 ins., measured from points at distances 522 and 545 respec-

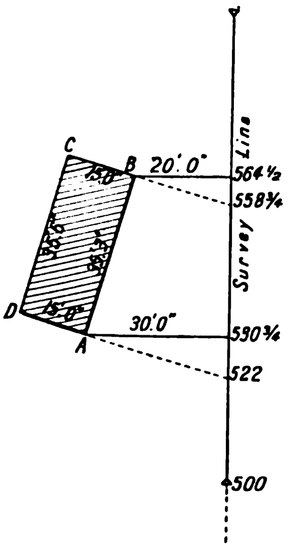

FIG. 37.—Offsets.

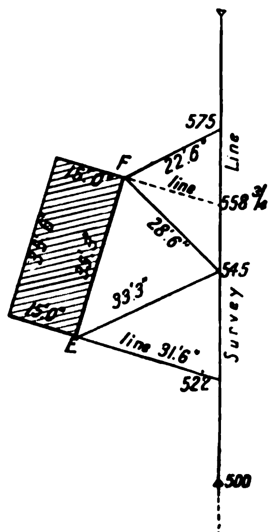

FIG. 38.—Ties.

tively along the chain line, and point F is similarly fixed by two measurements. To avoid recording too many points along the chain line, point 522 is chosen in line with the end of the house, and similarly point 545 is made to do double duty. The complete measurements which should be recorded are as shown in the figure.

A comparison of the two figures shows that the method by offsets involves less measuring on the ground and less sketching and figuring in the note book. The comparative accuracy of the methods will depend almost entirely on the accuracy with which the right angles in the offset method are measured. With a reliable optical square

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or prism properly used the offset method will be as accurate as the tie method, while, given equally accurate work in both cases, the plotting of offsets will involve less error than the plotting of ties and will at the same time be more expeditiously carried out.

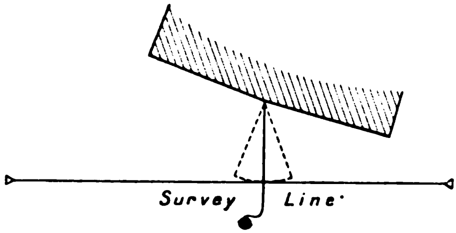

FIG. 39.—Finding Correct Offset Distance.

The extent to which the perpendicularity of right angle offsets may be estimated by the eye will depend in part on the scale to which the plan is to be plotted. On a scale of 20 ft. to the inch distances smaller than 3 ins. are scarcely appreciable on the paper, and for scales between 20 ft. and 80 ft. to an inch, it is customary to record chain distances, offsets and ties to the nearest quarter foot or half link. A person with an average eye will easily notice that one line is not perpendicular to another if the deviation amounts to 1 ft. in forty. This would indicate that the length of offset estimated by eye should not exceed 10 ft. if the error in location of the point is to be under 3 ins. For small scales, such as the $\frac{1}{2500}$, where distances under 2 ft. are scarcely appreciable on the paper, longer offsets may be set out by the eye. The method of procedure in finding the length and position of a perpendicular offset is as indicated in Fig. 39. The length of the offset is the shortest distance from object to chain obtained by swinging the tape about the object as centre. The position of the offset on the chain will, in general, be as accurately determined by noting the point where the tape makes symmetrical angles with the chain.

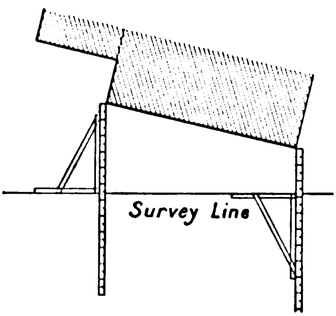

FIG. 40.—Accurate Short Offsets.

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The method of setting out and measuring accurate short offsets by means of a large wooden set-square and an offset staff, as sometimes practised in town surveying, is illustrated in Fig. 40.

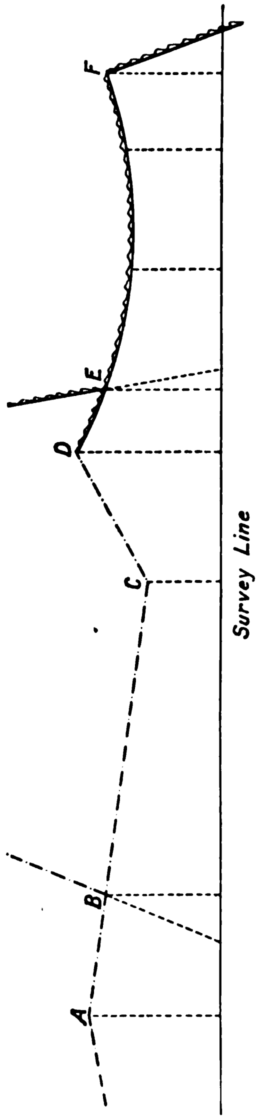

FIG. 41.—Offsets to Boundary.

Points to which Offsets or Ties should be Taken.—The position of any straight line relative to a survey line is absolutely determined if its two end points are correctly fixed by offsets or ties. Any line or boundary, therefore, which consists of a series of straights will be completely located by taking offsets or ties to all the angles. In Fig. 41 the straight portions of boundary AC and CD are completely determined by the offsets taken to the points A, C, and D. Any straight line, whose ends have been fixed, may be used as a subsidiary base line to which intermediate points and adjacent objects may be referred. For example, it may be more convenient to determine the point B on the straight line AC by a measurement of the distance AB than by an offset or by ties from the survey line.

Curving lines or boundaries are surveyed by taking offsets or ties to points chosen at intervals along the curve, the spacing being such that a fair curve drawn through the points plotted on paper will not differ appreciably from the true form of the boundary. The flatter and more regular the curve the greater may be the interval between offsets, while the larger the scale of the plan the smaller should be the interval. A railway curve of uniform curvature may be accurately drawn in, with the use of manufactured curves as rulers, from points surveyed at fairly wide intervals, provided particular care is taken to fix the end portions sufficiently minutely, as the curvature is often variable near the tangent points. In surveying regularly curving objects it is most convenient, both as regards booking and plotting, to take the offsets at equal intervals, say every chain length or every 20 or 50 ft. or links, or other round distance, as the case may require. In Fig. 41 positions of offsets are indicated for a curving boundary, DEF. Junction points with other boundaries, such as point E, must be carefully and accurately located, as well as all definite angles, such as point F.

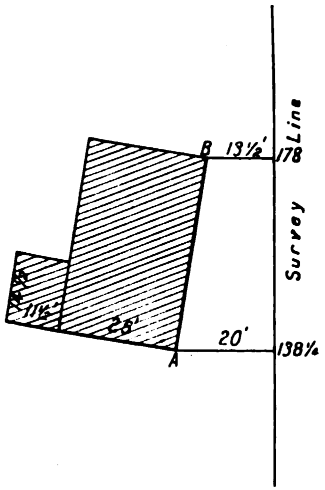

Fig. 42.—Fixing a Building.

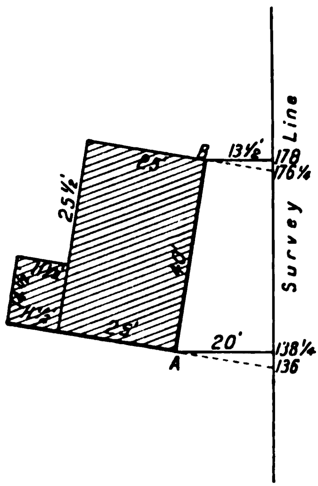

Fig. 43.—Complete Measurements to fix a Building.

Fixing Buildings, &c.—Detached buildings and houses commonly consist of rectangular forms in plan. In surveying such buildings, where they are of simple outline, it will usually be preferable to carefully fix one of the longer sides of the building in relation to the survey line and to locate the rest of the building in such a way that it can be plotted from this side as a base. Sufficient measurements must be taken to check the directions which the sides make with each other. Fig. 42 shows a rectangular building lying with a long side AB adjacent to the survey line. Points A and B would be fixed by perpendicular offsets or by ties. The further measurements necessary to enable the building to be plotted are the width

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of the main building and the length and width of the projection at the back. The plotted points A and B fix the long side of the building on the paper and furnish a base line from which the remaining sides of the building may be plotted.

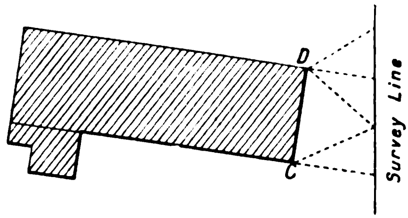

FIG. 44.—Fixing Building from Short Side.

The measurements shown in Fig. 42, if correctly made, would enable a rectangular building to be accurately plotted. A mistake in any of the measurements would, however, remain undetected. To check the accuracy of the work and to enable the surveyor to have confidence in his results, all the measurements shown in Fig. 43 should be taken and recorded. The measured length of AB then furnishes a check on the accuracy of the plotted points, and the true directions of the ends of the building are established by noting the points where these directions cut the survey line. The accuracy of the various sides is to a large extent proved by taking the separate measurement of each. It may lead to error to assume that corresponding sides are equal.

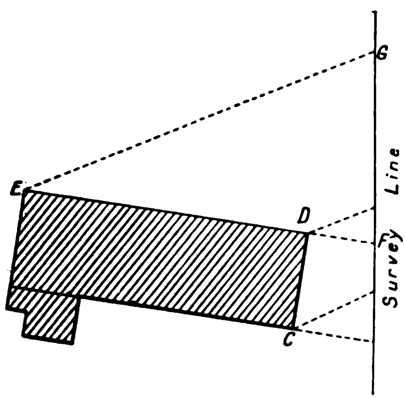

FIG. 45.—Fixing Building from Long Side.

If the side adjacent to the survey line is short in comparison with the side of the building it will not be sufficiently accurate simply to fix that side in relation to the survey line. The dotted lines in Fig. 44 indicate the measurements which are sufficient, theoretically, to completely fix the side CD and the directions of the two adjacent sides. The location of the building from these measurements would not, however, be reliable, as the length of CD is too short in comparison with the size of the building to furnish a proper base. A small error at C or D would cause a magnified

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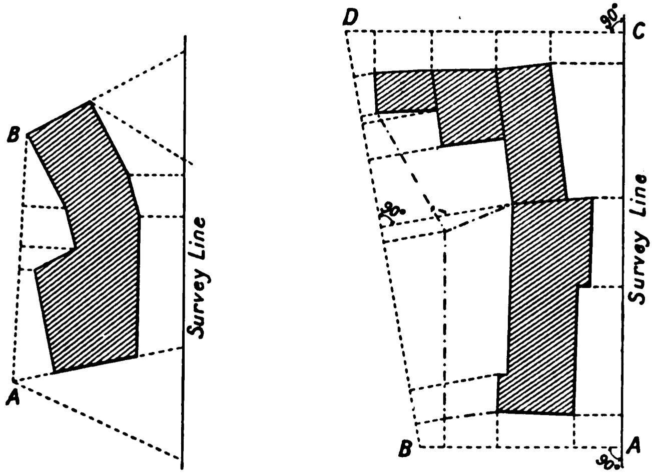
displacement of the other end of the building. It is, therefore, advisable in such a case to proceed as shown in Fig. 45, where the position and direction of the long side DE are fixed by the ties FE and GE. The rest of the building may then be plotted from DE as a base, with small possibility of error.

In the case of irregular-shaped buildings it will seldom happen that all the essential corners can be fixed directly from the survey line. The shape of the building will in many cases not be definitely fixed, even when all the corners adjacent to the survey line have been located by offsets or ties and the measurements of all the sides



FIGS. 46 AND 47.—Fixing Irregular Buildings.

have been taken. Corners which are unattainable from a main survey line will often be best fixed from a subsidiary survey line specially set out for the purpose. Under suitable circumstances subsidiary lines may be set out to encompass the building. Fig. 46 shows an irregular-shaped building and indicates a method by which it might be completely surveyed. The front corners and the ends of the building are fixed by the offsets and ties from the survey line. The tie lines used in locating the ends of the building serve to fix the points A and B. A line run between these points serves as a subsidiary survey line, from which to fix the remaining corners by offsets or ties. The measured lengths of the sides of the building form a check on the accuracy of the plotted points.

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Fig. 47 shows how the optical square may be utilised in arranging a system of subsidiary lines to enclose an irregular building. Lines AB and CD are set off at right angles to the survey lines and serve along with the line run from B to D as base lines, from which all the essential corners can be located. In the figure the various corners are shown fixed by means of perpendicular offsets. In practice, the methods of fixing the points will depend largely on the circumstances of each case. A point which, owing to some obstruction of view or otherwise, cannot be determined by a perpendicular offset may often be readily fixed by ties and *vice versâ*.

Locating Irregular Boundaries, &c.—A portion of an irregular boundary may consist of short lengths of straight. In that case, as already indicated, offsets would be taken to each corner formed by the junction of two straights. In surveying boundaries which

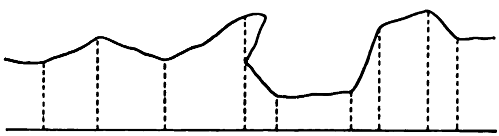

FIG. 48.—Offsets to Irregular Boundary.

are completely irregular, as indicated in Fig. 48, the same principle is applied. Points of abrupt change are chosen, dividing up the boundary into portions which, apart from minor irregularities, may be considered as sensibly straight. Offsets are taken to these points, and the forms of the intervening sinuosities are sketched in the note book as accurately as possible.

The amount of care which should be expended in the location will depend on the nature of the boundary. In surveying definite and permanent property divisions, however irregular, the offsets and measurements should be so taken that there will be no room for appreciable variation between the plotted form and the actual. In the case of variable outlines, such as the margin of rivers and lakes, whose position depends on the level of the water, and likewise in the case of indefinite boundaries, such as the edge of marshes, &c., while the broad features of the boundary should be accurately determined, there is no need for great refinement in fixing minor details.

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Length of Offsets.—The closer the chain line lies to the objects and features which are to be surveyed, the more accurately and expeditiously will the work be accomplished. The ordinary linen tape with which offsets and detail measurements are made is not a very reliable instrument. An error of 3 ins. is not uncommon in a tape 66 ft. long, and hence one reason for limiting the length of offsets to as small a size as possible. Measurements which exceed one tape length cause extra trouble and delay, so that the survey line should preferably never be so far from the work as to cause the length of a tie line or offset to exceed the length of the tape. The aim should always be to make the offsets as small as possible, and it will often be true economy to lay out some additional survey lines to effect this. A case in point often occurs in surveying a wide road with detached houses on each side. A single survey line might, with the use of long offsets and ties, suffice to pick up both sides of the road and the fronts of the houses. It would be generally preferable, however, even apart from considerations of interference due to traffic, to lay out a survey line on each side of the road and to survey each side separately. The booking of the notes is thereby simplified, and likewise the plotting, and there is less chance of omission and error.

Running a Survey Line.—The methods of ranging out a line and measuring its length have been already explained. In running a survey line, that is, in performing simultaneously the operations of chaining the line and locating objects from it, the chain is left lying lined out straight on the ground at each length until all the necessary offsets, ties, and other measurements have been taken. Where several offsets or ties require to be measured from the same chain length, they should be taken systematically in order, working from the rear towards the forward end of the chain. Mistakes in reading the chain and in booking are apt to occur if the offsets are taken out of consecutive order or working backwards. Where numerous offsets occur on each side of the chain, it is sometimes advisable to take first all the offsets and measurements on one side of the chain and then all those on the other side. Figs. 51 to 53 show some examples taken from actual practice of lines run in surveying various commonly occurring classes of features.

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They give an idea of the offsets and measurements required and the methods of booking.

Field Book.—The main requirements of the field note book are that it should contain good quality stout opaque paper, should be well bound and of a size convenient for the pocket. Common sizes for field books run from 7 ins. by 4½ ins. to 9 ins. by 5½ ins. The paper may be either plain or ruled with a faint red line down the centre of each page to represent the survey line. Instead of a single line two parallel lines a short distance apart are sometimes used, the column thus formed being reserved for the insertion of all distances along the survey line. Any chance of confusion between these distances and the offset or other measurements is thereby avoided. The plain paper field book can be utilised for

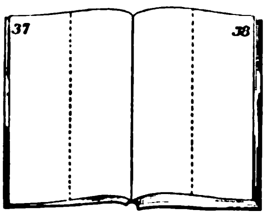

FIG. 49.

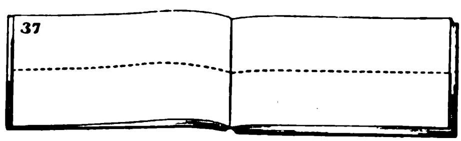

FIG. 50.

all kinds of note-taking, and is more convenient than the ruled form if a large amount of general sketching is required. When a survey line is to be booked a pencil line is ruled on the paper to represent the survey line. If a considerable width of ground is being surveyed all on one side of the chain, it may be advisable to rule the pencil line near one edge of the page so as to get sufficient room for the notes. Where the bulk of the work consists of recording the details of survey lines the note book with single ruled line is recommended. The book may open either the short way or the long way, as indicated in Figs. 49 and 50. In the latter case the line is continuous from side to side and the two pages become practically one, and are usually so numbered.

On the inner front page the surveyor's name and address should be written. The date and the names or initials of the members of the party should be inserted at the top of each page on which a fresh day's work is commenced. A few pages at the beginning of the book should be reserved for reference diagrams of the survey lines and an index giving the page or pages on which each line is detailed.

Field Notes.—The importance of making the field notes clear, accurate, and complete cannot be too much emphasised. A good quality pencil should be used, kept well pointed. It should not be so soft as to smear the paper when the hand is brushed across the page, nor so hard as to make but faint marks. A pencil of degree H or F will usually be found suitable. Figures should be plain and well formed. Whenever dubiety would be likely to arise as to the points between which a figured distance extends, the proper points should be clearly indicated by means of arrow heads or otherwise. The clearness and neatness of the work is greatly enhanced if all notes, names and explanations, &c., are neatly printed in italics or block. With practice neat sloping italic printing can be done almost as quickly as writing. The sketches should be arranged of a size such that there will be ample room for all figures. Crowded figures are a fruitful source of confusion and mistakes in plotting.

Without attempting to draw the objects surveyed to scale the endeavour should be made to have them set down in fairly recognisable proportion and in good relationship to the survey line. Straight lines should be shown straight, all definite angles and corners should be clearly indicated as such, points where a straight outline changes to a curving one should be carefully marked, and the sketch of an irregular boundary should be as true to actual form as possible. It will be found advantageous in sketching to make the following exaggerations as compared with a true to scale plan: Magnify the size at points of minute detail, allowing in every case sufficient room for the dimensions. Decrease the length of all long straight lines. Where two straight lines meet at an angle of nearly 180°, so as to be nearly in one line, draw them at a more acute angle in the sketch and make the position of the corner quite definite. A fence or boundary line, &c., which crosses the survey line nearly, but not quite at right angles, should be drawn on the sketch with an exaggerated inclination to the perpendicular in the proper direction. When, as sometimes happens, it is unavoidable that an actual straight line should be represented on the sketch

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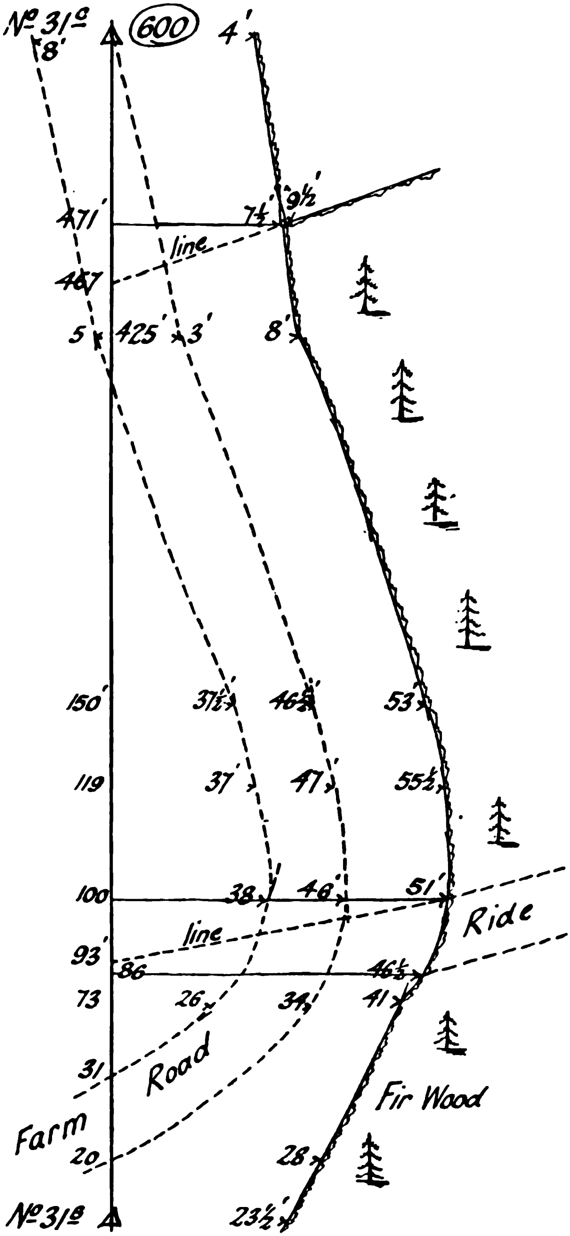

FIG. 51.—Example of Survey Line.

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by a crooked line, write the word "straight" alongside and indicate the extent by arrow heads. This is most often necessary where straight lines cross the survey line.

Examples of Survey Lines.—Fig. 51 shows a page extracted from a survey book to illustrate the method of booking perpendicular offsets. It is typical of much of the kind of surveying required in open cultivated country. The straight line running up the page represents the survey line, the commencement being at the bottom of the page. The objects to be surveyed are sketched about this line in the position which they occupy relative to the survey line and the distances along the survey line, and the lengths of the perpendicular offsets to the defining points are figured on. Where offsets to several points are taken from the same point on the chain line, the distance to each point should be measured from the chain. Thus, in the figure, where three offsets are shown from the same point on the chain, the lengths figured are the total measurements from the chainline to each point, not the separate distances from point to point.

Fig. 52 shows the notes of a survey line on a curving railway embankment. The stations are fixed clear of the running lines, so that poles may be planted for lining out purposes without risk of disturbance from trains, and in order that the theodolite, if used, may be safely set up. In railway work the boundaries, railway lines, and main structures are of primary importance, while edges of slopes and such like are of less importance and need not be surveyed in great refinement. Since the rails in a double track are parallel it is only necessary to survey one of the rails and to measure once and for all the widths of the spaces between the rails. To facilitate plotting, the offsets should be taken to the same rail throughout the length of each survey line. When the surveyed rail has been plotted on the plan the others which are parallel to it may be drawn in at the correct distance apart.

Fig. 53 shows the notes of a survey line locating an irregular stream and boundary fence. The offsets to fix the stream are taken at the points which best determine the changes of direction of its banks. Minor irregularities between offset points should be recorded by careful sketching only.

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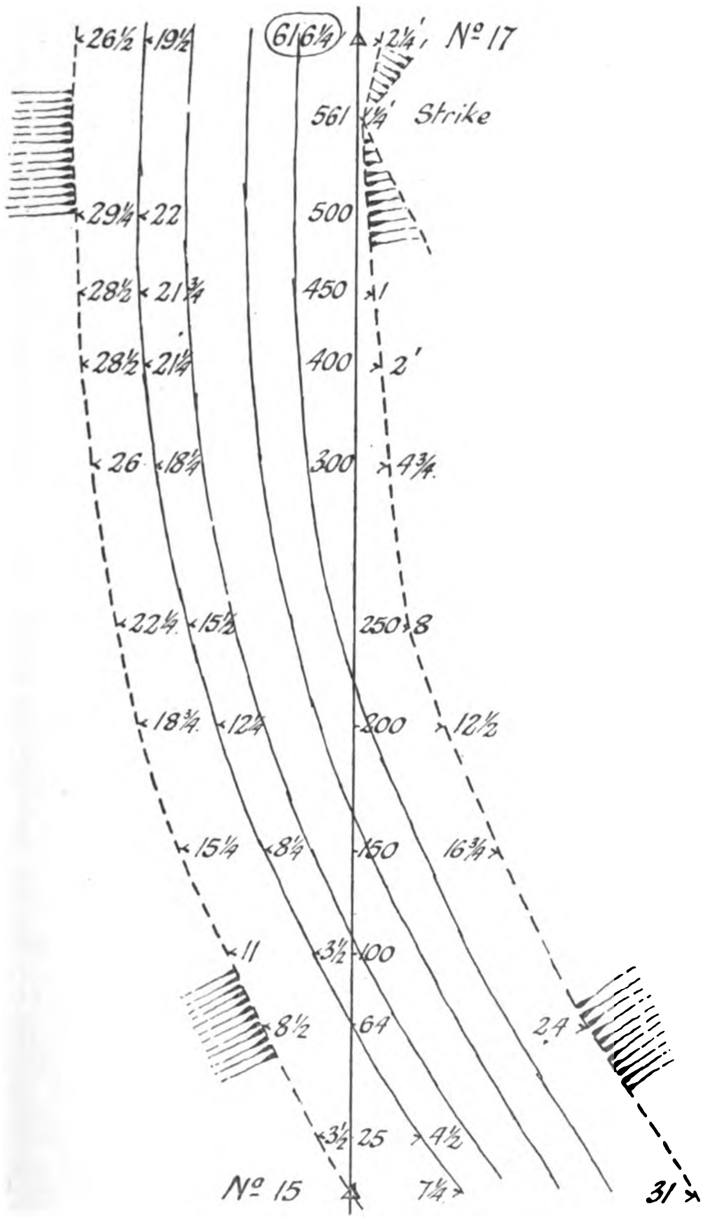

Fig. 52.—Example of Survey Line—Railway Lines.

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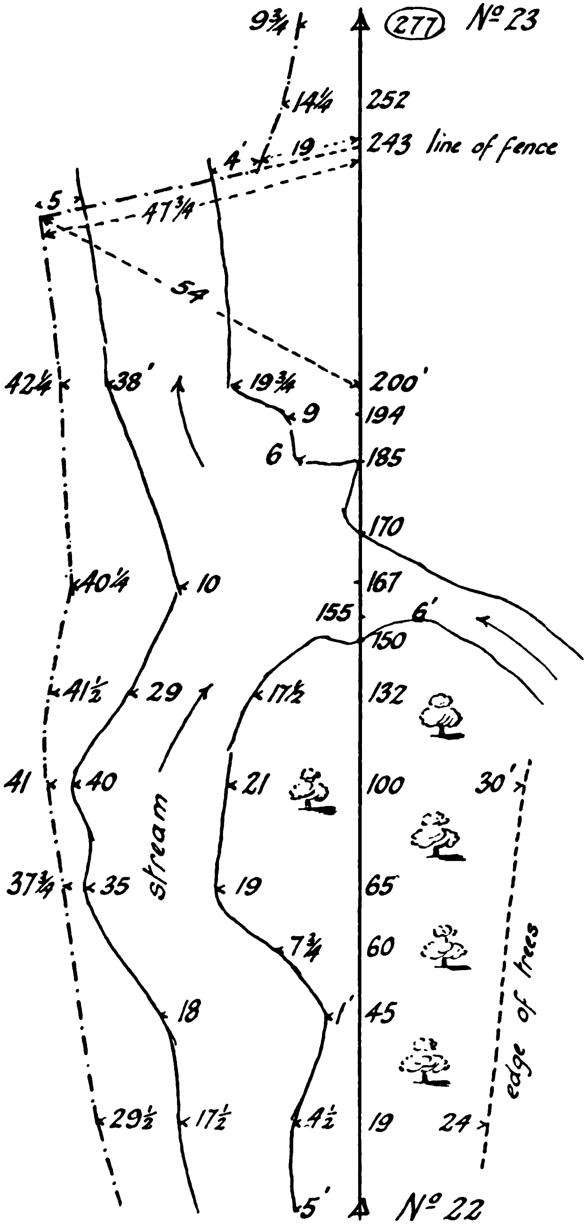

FIG. 53.—Example of Survey Line—Stream, etc.

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CHAPTER V

CHAIN SURVEYING—ARRANGEMENT OF SURVEY LINES

The principles governing the laying out of a system of lines suitable for surveying a small area are dealt with in this chapter, and examples of desirable arrangements of lines are given. No hard and fast rules as to arrangement of survey lines can be given, as this is largely controlled by the configuration and features of each area.

Arrangement of Survey Lines.—The framework of survey lines forming the basis of a chain survey must be laid out as a connected system of triangles. To get good results in plotting, the triangles should be as nearly equilateral as possible. To exclude the possibility of large error due to ill-conditioned triangles, it should be made a rule that no angle of a triangle should be less than 30°.

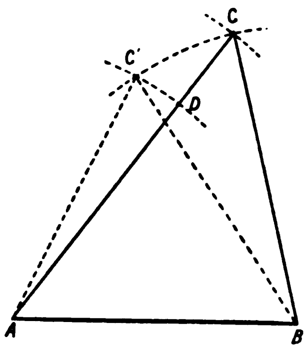

Fig. 54.—Displacement of a Point due to Error in Side.

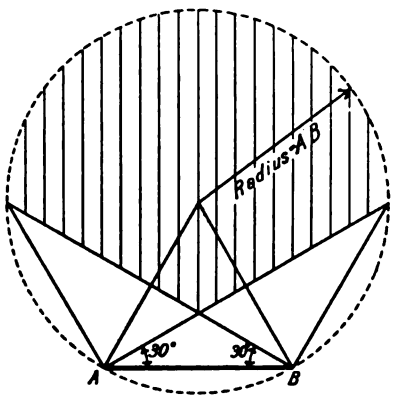

Fig. 55.—Limits of Well-conditioned Triangle.

Let A and B (Fig. 54) represent two points plotted in correct relation to each other, and let C represent the correct position of the third point, whose position has been fixed by measuring the

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distances AC and BC. If, due to a mistake or error, the length AD has been used instead of the true length AC, the plotted point will be at C′ at a distance C′C from its true position. An inspection of the figure shows that for an error confined to one of the sides the resulting displacement of the plotted point can never be less than the error. Practically, if the angle at C is 90° the displacement of C will be just equal to the error in the side AC; for an angle of 60° the displacement will be 1·15 times the error, and for an angle of 30° the displacement will be equal to twice the error. For angles smaller than 30° the displacement will be still greater. If the chance of error is confined to two sides of the triangle the best result will be obtained when these sides make an angle of 90° with each other. If, however, there is equal liability to error in all

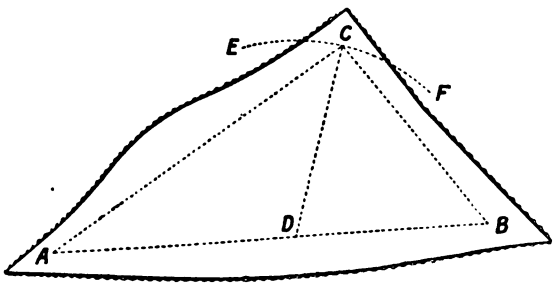

FIG. 56.—Proof Line for Triangle.

three sides of a triangle, the best form is the equilateral. The condition that no angle of a triangle should be less than 30° (which necessarily involves that no angle should be greater than 120°) means that the third angle of a triangle on the base AB (Fig. 55), may be anywhere within the hatched area.

The endeavour should be to have as few main triangles in the framework as possible and to have them well checked by proof lines. A proof line is a line which serves to test or rather confirm the accuracy of a triangle. Take the case of a single triangular field surveyed by a single triangle, as shown in Fig. 56. A mistake in the measurement of any of the sides would cause the field to be wrongly plotted, and the mistake would not be detected unless a check measurement be made. The measurement of the length of the line from C to a point D fixed by measurement along the line AB would form such a check in the case of the given

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triangle. If after plotting the triangle ABC the scaled length of CD is found to agree with the length measured on the ground, then it may reasonably be assumed that the surveying and plotting are correct. It should be clearly noted that a proof line, such as CD, does not form an absolute test of the accuracy of the work, as any errors in the measurement of the sides which caused a displacement, however considerable, of point C along the arc EF struck from centre D, would remain undetected.

The main survey lines should be arranged to pick up as much of the detail as possible, and should, as far as practicable, approach most closely to the more important portions of the work. The condition as to well-shaped triangles may require some of the main lines to be set out without regard to picking up detail, and such

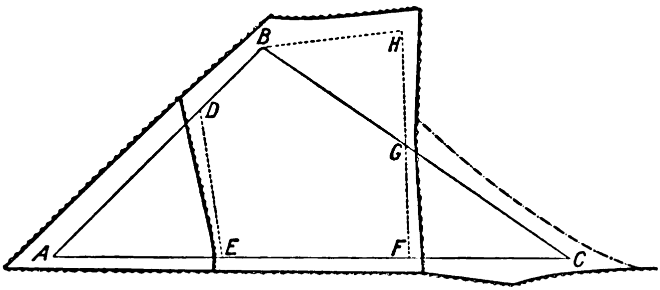

FIG. 57.—Single Triangle forming Basis of Survey Lines.

lines should be arranged with due consideration to the necessity for connecting subsidiary lines to them. An existing plan of an area to be surveyed is of considerable service in the preliminary work. The proposed main stations being marked on roughly in position, it can be seen at a glance whether the system of triangles is good or indifferent. It is an instructive exercise for the beginner in surveying to take a good existing map of a small area, and, without going to the ground, lay down thereon a system of lines which in his judgment would be suitable for surveying the features shown. Let him then proceed to the area and examine whether his lines are practicable. It will be surprising if his arrangement at all suits the actual conditions, and the comparison should impress on him the extent to which the layout of survey lines is governed by the configuration of the ground and minor natural features which affect the lines of view.

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Fig. 57 indicates how a single main triangle may serve as the basis for surveying an area of several enclosures. The subsidiary lines DE and GF check the accuracy of the main triangle, and at the same time serve to locate the interior boundaries. Line FG

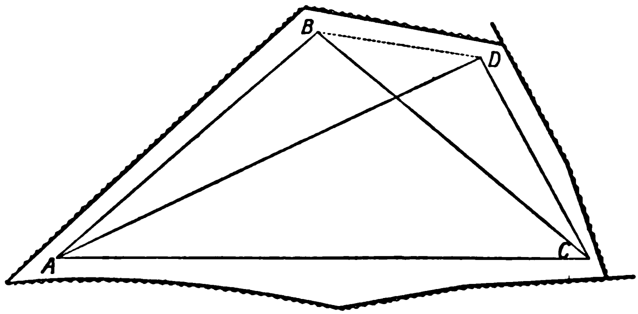

Fig. 58.—Survey Lines for Quadrilateral Enclosure.

is produced to H, and the accuracy of this point is checked by BH.

To survey a quadrilateral enclosure two triangles will usually be required. In the case illustrated in Fig. 58 the triangles which would be plotted are ABC and ADC. Line BD forms the check

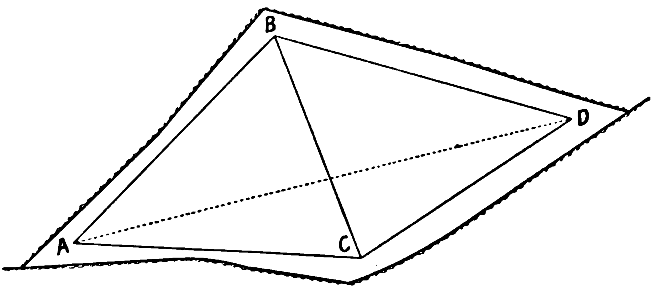

Fig. 59.—Survey Lines for Quadrilateral Enclosure.

line. Instead of proceeding as above, point D might be determined by laying down the triangle BDC on the base line BC, but as BDC is a badly conditioned triangle the accuracy of the plotting would not be reliable. Similar considerations would indicate in the case of Fig. 59 that the triangles ABC and BDC should be plotted, the diagonal AD serving as check line.

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For more extensive surveys an arrangement of triangles laid out on each side of a straight line running through the area is to be desired. Such an arrangement is shown in Fig. 60. Line AB represents the through base line. This would be laid down first on the paper. The triangles shown in full lines would then be plotted. Check lines and subsidiary survey lines are shown dotted.

Marking Survey Stations.—The survey stations require to be marked in a more or less permanent manner, depending on the extent and duration of the survey. In soft ground wooden pegs, and in hard roads or streets pointed iron spikes or nails are most

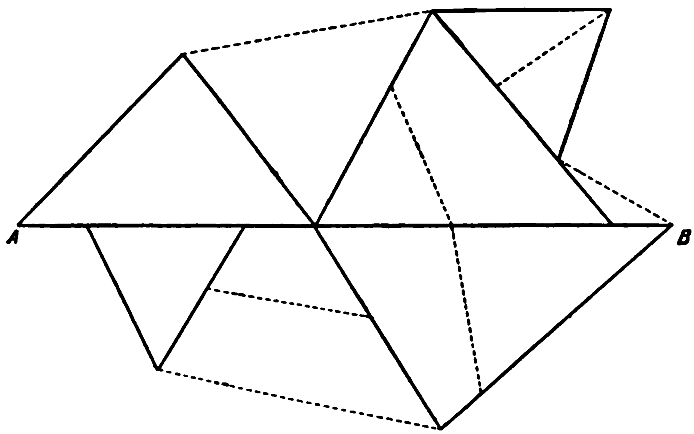

FIG. 60.—Arrangement of Triangles.

convenient for the purpose. They should be driven till flush with the surface wherever there is traffic. Wooden pegs may be rendered more conspicuous and easily found by cutting away the turf around their heads.

Referencing Survey Stations.—The principal stations should be carefully "referenced," so that their positions may be recovered should the pegs themselves happen to get removed. A station may be referenced by taking accurate measurements to it from two definite points on adjacent permanent objects. The more nearly the lines joining the peg to the reference points intersect at right angles, the more definitely will the station be fixed. Stations at a distance from any permanent objects may be referenced by driving

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two additional pegs, one on each side of the station and some distance away from it, and so that all three pegs are in line. The distance from each side peg to the station is noted. To avoid confusion the reference pegs should be distinct in character from the station peg.

Numbering Stations.—A diagram of the survey lines with main stations numbered should be inserted in the beginning of the field note book. Minor stations along the main lines, which are usually fixed as the work proceeds, may be given the number of the nearest main station with a distinguishing letter, as 9A, 13C, &c. Each survey line, as booked in detail in the field book, should have the numbers of the commencing, terminal and intermediate stations clearly marked to correspond with the numbers on the reference diagram. A line commencing, say, at station 6 and terminating at station 7 is designated Line 6—7, and this should be plainly written or printed at the top of the page containing the notes of this line, and similarly for all the lines. It is a convenience to have the lengths of all main lines marked on the reference diagram.

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CHAPTER VI

CHAIN SURVEYING—ERRORS

ONLY from a careful study of the errors which may arise in the various operations of surveying and the relative magnitudes of these errors can a surveyor attain to a proper appreciation of the relative importance of the operations and be enabled to arrange his work to the best advantage, having due regard to accuracy and economy. This chapter deals with the principal errors which may arise in the operations of chain surveying, including errors in measuring the lengths of lines resulting from various causes, errors in ranging out the survey lines, and errors in locating objects and details.

Errors in Measuring a Length.—The important sources of error in linear measurements are :—

(*a*) Incorrect length of chain and incorrect graduation.

(*b*) Chain not held horizontal, or wrong allowance made for slope.

(*c*) Chain not stretched straight and tight between its ends.

(*d*) Sag on chain.

(*e*) Arrows wrongly fixed at end of chain. Chain not correctly held against arrow.

(*f*) Mistakes in reading the chain and in recording distances.

Error in Length of Chain.—A new steel band should be tested on an official standard. At a temperature of 60° and under a pull of 15 lbs. the error of a steel band should not exceed $\frac{1}{4}$ in. Excluding the slight alterations due to variation of temperature and pull, a band may be considered as of constant length for all purposes for which a chain survey is likely to be required. A very slight lengthening may occur, due to wear at the joints of the handles, or from splicing a break.

The steel chain, as already indicated, is of very variable length.

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A given pull causes a much larger stretch than in the case of the band, and alterations due to variation of pull may be a considerable source of error. The chain when in use continually changes in length, and these alterations are an important source of error.

It is important to distinguish between the effect of a constant error and that of a variable error on the accuracy of a survey. Suppose that a survey has been made with a band 101 ft. long divided into 100 equal parts supposed to be feet. The error in the length of the band will not affect the plotting or consistent accuracy of the work. If the scale of the plan be drawn on the assumption that the band is 100 ft. long, all measurements made by this scale will be wrong to the extent of 1 ft. in every 100 ft. If, on discovery of the error in the band, the scale be altered so that the distance which formerly represented 100 ft. be now made to represent 101 ft. the plan will be as accurate as if it had been made with a correct band.

The whole effect of a constant error in the measuring instrument is to cause the scale of the plan to be slightly wrong. If the extent of the constant error is known, the scale can be altered proportionately so as to make the plan correct.

The case is different if one portion of a survey be made with an incorrect chain and another portion with a correct chain. It will not now be possible to plot a correct plan, as the two portions are inconsistent. The same result is produced if the measuring instrument, such as the steel link chain, is subject to variable error, that is if its length alters during the course of the work. In important work, if the error is to be kept to the lowest possible limit, the chain should be made correct before starting each day's work, checked on a reliable standard at the end of the day, its error, if any, noted, and when necessary a variable correction applied to the lengths chained during the day, proceeding on the assumption that the error has increased gradually.

It should be carefully noted in making the corrections that if the chain is too long, all lengths measured will be too short, while if the chain is too short, the measured length of any distance will be too great. Inattention to this may result in the error of a length being doubled, in the belief that it is being eliminated.

Rule to find the correct length of a line measured with an incorrect chain or tape :—

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Multiply the measured length by the ratio of the incorrect length of the chain to the correct length.

Suppose that the distance as measured between two points with a chain which was $\frac{3}{4}$ in. too long, was 788·3 ft. To find the true distance—

$$\text{Ratio} \quad \frac{\text{Incorrect length}}{\text{Correct length}} = \frac{100 \cdot 06}{100} \qquad 1 \cdot 0006.$$

True distance $= 788 \cdot 3 \times 1 \cdot 0006 = 788 \cdot 8$ ft.

The above result will be correct if the chain is graduated in even divisions. If the chain is irregularly graduated the total error will consist of two portions—one due to the error in length of the chain and proportional to the number of whole chain lengths measured, the other due to the incorrect graduation and occurring in the final fraction of a chain length. Graduation errors might be positive at one part of the chain and negative at the other.

Error due to Chain or Tape not being stretched Horizontal.—Any measurement taken on the slope is greater than the true horizontal measurement. The result, therefore, of not holding the chain truly horizontal, or of neglecting to allow for the effect of slope, is to cause the measurement to be too big. The error in a distance measured along a uniform slope will be proportional to the distance. As no surface other than a level one can have the same slope in all directions, the general result of neglecting slope in chain surveying will be to introduce variable error (always positive) in the lengths of the lines, with adverse effect on the accuracy of the work.

The tables in Chapter III. show the extent of the error introduced in measuring on various slopes. The limit up to which a slope may in practice be treated as level can be determined from an examination of the table, keeping in view the purpose of the survey and the accuracy required.

Errors from Chain not being stretched Straight and from Sag on Chain.—These two sources of error give rise to effects similar to those produced by neglecting slope. The error is always positive, that is, the measured distances are too great. With a 100-ft. chain a sag of about 8 ins. below the line joining the ends, or a like horizontal deviation at its middle when lying on the ground, will cause

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an error of $\frac{1}{8}$ in. When ordinary care is exercised in straightening the chain there is no room for appreciable error due to this cause. The case where the chain requires to be stretched across a wide opening with considerable sag seldom occurs. As the length, especially with a link chain, is considerably affected by the increased pull required, the most satisfactory method of getting the true length is to note the amount of sag and afterwards hang the chain with the same amount of sag in a position where the horizontal distance between its ends can be accurately measured. This can be readily effected on the side of a wall.

Errors in Fixing Arrows and Marking Chain Lengths.—On level ground, where the handles can be held down on the surface, there should be little error in fixing the arrows or otherwise marking the chain lengths. When, however, owing to irregular ground, rank growth, slope, &c., the handle cannot be held to the ground the

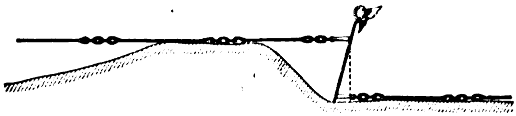

FIG. 61.—Error from Careless Fixing of Arrow.

chance of error is greatly increased. If the leader, holding the top of his arrow against the handle, sticks it into the ground somewhat off the perpendicular, and the follower thereafter holds his handle to the bottom of the arrow, an error will be introduced, as indicated in Fig. 61. An error will also arise if the follower moves the head of an arrow against which he may require to hold. Where, owing to slope, &c., the chain handle requires to be held some distance above the ground, careful use of the plumb-bob is necessary if error in marking the true distance on the ground is to be avoided. Some error is inevitable if the rough and ready method of dropping stones or arrows is employed.

The error in marking chain lengths may be positive at one chain length and negative at another, and on a line of considerable length the positive and negative errors will partly balance. The error is said to be compensating.

Errors and Mistakes in Reading the Chain or Tape.—Such errors are usually only apt to occur once in measuring a given length,

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namely, in reading the final fraction of a chain length. The mistakes liable to occur through reading the chain backwards and through reading on the wrong side of the decimal divisions, as 29 ft. for 31 ft., have been already referred to. In using the cloth tape beginners are apt to read 6 for 9 and *vice versâ* if they happen to hold the tape with the figures upside down. The same cause sometimes leads to 32 being called out where the correct distance is 23. To avoid mistakes such as the above one must know where they are likely to arise and be constantly on the look out for them.

Errors in Ranging out Survey Lines.—The effect of any deviation of a survey line from the straight likely to occur in ordinary work may be entirely neglected as a source of error in the *length* of the line. Careless lining is, however, an important source of error in the *position* of details located from a survey line. A long line chained without the use of intermediate ranging poles as guides will usually deviate somewhat from a straight line. The same is to be expected with any line ranged out by the eye over a hill or across a valley. A survey line though curved on the ground will be represented by a straight line on the plan, and hence any objects located from the survey line will be out of correct position on the plan by the extent of the deviation. In uneven country this forms one of the principal sources of inaccuracy in chain surveying and is almost unavoidable when the unaided eye is used to range out the lines.

Errors in Locating Objects from Survey Lines.—An error in the length of a survey line will affect the accurate plotting of all other survey lines subsequently connected to it. An error in an offset or tie from a survey line will usually only affect the plotting of one point, or at most, a small portion of detail. The important factor in determining whether some of the ordinary sources of error in measurement need be reckoned with at all in the case of offsets and detail measurements, is the scale of the plan. A careful draughtsman can plot to $\frac{1}{100}$ in., so that on the scale of 1 in. = 100 ft., the smallest distance which can be definitely plotted and scaled will be one foot. It would be manifestly absurd in such a case to take the detail measurements to the nearest inch. The offsets and ties might in practice be taken to the nearest half foot and still leave a margin sufficient to cover ordinary errors of

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measurement, such as inaccuracy of tape, tape not held quite horizontal, &c.

In working to a scale of 1 in. to 40 ft. it would be appropriate to read to the nearest 3 in. or half link. To the scale of $\frac{1}{2500}$, which is about 1 in. to 208 ft., measurements need only be recorded to the nearest foot. The limitations as to the general accuracy attainable in chain surveying render futile any attempt to read to closer than 3 ins. in the detail measurements.

The foregoing considerations indicate that with ordinary care errors in the *length* of offsets and ties should not be so large as to affect the accuracy of the survey. The chief sources of error in locating details are to be found in offsets not taken truly perpendicular, ties forming ill-conditioned triangles, and mistakes in reading the tape.

Mistakes in reading the tape and mistakes in booking the measurements are the most serious sources of error in locating details. Mistakes of the former class have already been referred to under errors in measuring a length. Mistakes due to misapprehending the figure called out by an assistant will be practically avoided if the surveyor repeats the figure aloud. The assistant will then correct him if he finds that the figure has been wrongly taken up. A surveyor will sometimes write down a wrong figure though he has the correct one in his mind. This is rather liable to happen if another distance is called out while he is writing the previous one down.

The largeness of the mistakes in the above classes often leads to their detection during plotting, but they may often remain undetected. Small mistakes of a few feet are most dangerous, as they are likely to pass unnoticed and to remain as errors in the survey.

Permissible Error.—The accuracy attainable in chaining is very largely dependent on the nature of the ground. Under equal conditions the steel band gives much better results than the steel chain. An error not exceeding $\frac{1}{2000}$ or 1 ft. in 2,000 ft. may be easily attained with a steel band on uniform ground without special precautions. It may be very difficult to keep within an error of $\frac{1}{1000}$ using a steel chain on undulating ground. In the latter case it has to be recognised that the attainable accuracy is in large

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measure limited by the imperfections of the human eye as an instrument for ranging straight lines. There is thus a considerable extent of unavoidable error, and it is not profitable to spend much time and trouble in eliminating sources of error which are very small in comparison. The main endeavour should be directed towards attaining the degree of precision required for the purpose in hand, and to this end most attention should be paid to the eliminating of those sources of error which are relatively largest.

Expedition in Surveying.—The time taken in plotting a survey usually bears no inconsiderable proportion to the time spent in the field, and may sometimes be quite as long. Anything, therefore, which facilitates and shortens the plotting may as surely further the expedition of the work as anything which tends to curtail the duration of the field operations.

For good progress in the field the first essentials are a well-arranged system of survey lines and method and order in the field operations. Time spent in adjusting the survey lines so as to avoid difficult ground and lie close to the main portions of the work will usually be well repaid. It is in general unwise to attempt to locate a large extent of detail from one survey line. If the work is complicated time may be wasted in adjusting and altering the field sketches, and necessary measurements are apt to be omitted, causing much trouble and waste of time. The running of an extra survey line at complicated portions so as to bring the lines closer to the work and render them less complicated will often be amply repaid in the resulting simplicity, clearness and completeness of booking and ease of plotting.

If speed in running the survey lines is to be attained the assistants must be made to understand their duties and the system and order of operations. In many cases the desired results in surveying, as for example in the location of details, can be obtained in several different ways. An interesting and instructive survey might be carried out in which all the different methods of executing the various operations were illustrated, but such a survey would not be an expeditious one. Operations to effect a given result should as far as possible always be carried out in the same manner, that method being chosen which is simplest and best suited to the circumstances of the survey. With work thus arranged and

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standardised the assistants after a short experience will know without instruction from the surveyor what measurements to take and how to take them throughout much of the routine work. The surveyor must take pains to make his instructions to his assistants definite and explicit, especially so in regard to any unfamiliar procedure. Instructions wrongly executed through being misunderstood may give rise to much annoyance and delay.

Care and patience are well expended in cultivating and acquiring a good style of note-keeping. Badly formed figures, indefiniteness in marking the points between which a figured distance extends, and crowded and confused notes are inimical to speedy and accurate plotting.

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CHAPTER VII

CHAIN SURVEYING—SPECIAL PROBLEMS

This chapter deals with the execution of certain operations and the solution of some special surveying problems without the use of instruments for measuring angles. The operations and problems include setting out a right angle, dropping a perpendicular from a point on to a chain line, setting out a parallel line, setting out a given angle, chaining past an inaccessible area, chaining past obstacles, finding distance across a river, surveying far side of river, surveying pond, wood, &c., chain surveying in towns.

Setting out a Right Angle.—Three methods of setting out a right angle without the use of any instrument other than the tape or chain are shown in Figs. 62, 63, and 64. It is required in each

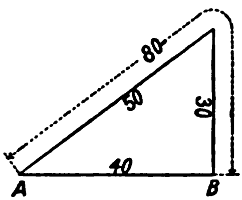

Fig. 62.

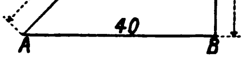

Fig. 64.

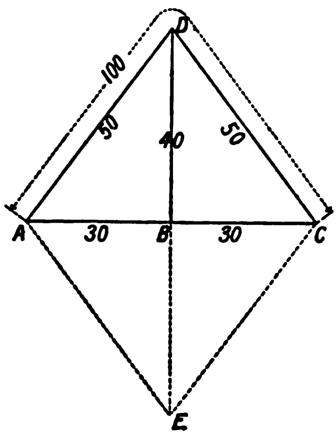

Fig. 63.

Setting out a Right Angle.

case to set out from point B a line perpendicular to the line AB. A triangle, the lengths of whose sides are proportional to the numbers 3, 4, and 5, is a right-angled triangle, since $5^2 = 3^2 + 4^2$. The application of this principle in setting out a right angle is illus-

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trated in Figs. 62 and 63. In Fig. 62, AB is measured off equal to 40 ft. One end of the chain is then held at A and the mark at 80 ft. is held at B. An assistant then takes the 50 mark and pulls

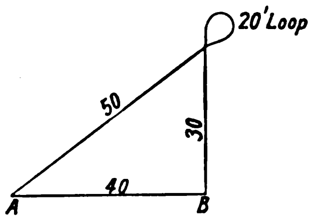

Fig. 65.—Right Angle by Steel Band.

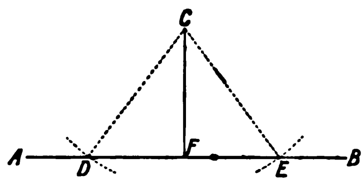

Fig. 66.—Perpendicular from a Point.

the chain out taut, thus forming a triangle with sides 40 ft., 50 ft. and 30 ft. long. The side 30 ft. long is then at right angles to AB. If the chain used is a band it will be better, in order to avoid a sharp bend in the chain, to hold one handle at A and the other handle at B and then to pull the chain taut while bringing the 50 mark to coincide with the 30 mark, as shown in Fig. 65.

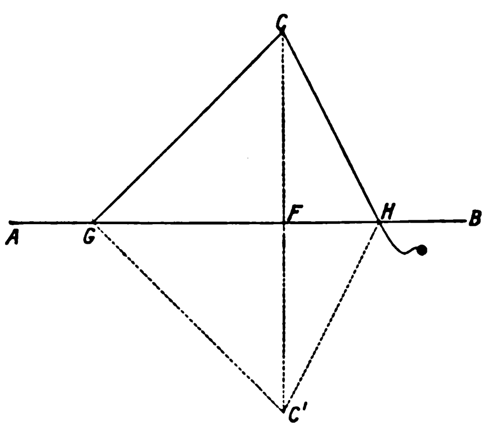

Fig. 67.—Perpendicular from a point.

In the method of Fig. 63, AB and BC are each set off equal to 30 ft. One end of the 100-ft. chain is held at A and the other end at B. The chain being then pulled taut from its centre point D, a line joining the latter to point B will be at right angles to AB. A more accurate perpendicular will be obtained if the above process is repeated to find a point E on the other side of AB. The line joining E to D will be 80 ft. long and at right angles to AB.

Another very useful arrangement of lengths forming a right-angled triangle is shown in Fig. 64, the sides being 40, 42 and 58 ft. long.

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Dropping a Perpendicular from a Point on to a Chain Line.—It is required to drop a perpendicular from point C (Fig. 66) on to the chain line AB. Take a length CD on the tape as radius, considerably greater than the perpendicular length CF. Swing this length about C as a centre and find the points D and E in which it cuts the chain line. Take point F midway between D and E, then CF is perpendicular to AB.

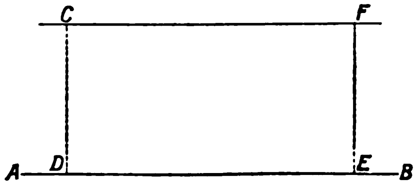

FIG. 68.—Setting out a Parallel Line.

Another method is illustrated in Fig. 67. Any two convenient points G and H are taken on the chain line, one on each side of the required perpendicular. A tape is stretched from G to C and on to H. Keeping the tape fixed at G and H, the point on it which touches C is taken hold of and pulled over to the other side of the chain line to C′. The line joining C to C′ is at right angles to line AB and cuts it in the required point F.

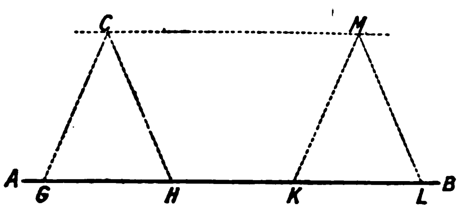

FIG. 69.—Setting out a Parallel Line.

Setting out a Parallel Line.—A line is to be set out through point C (Fig. 68) parallel to the line AB. Drop a perpendicular CD to the line AB and measure its length. At a convenient point E lay off EF perpendicular to AB and equal in length to CD. The line CF is parallel to AB.

If an optical square or other instrument for setting out a right angle is not available, it will be better to proceed as in Fig. 69. Points G and H are chosen on the chain line to give a well-proportioned triangle with point C, and the

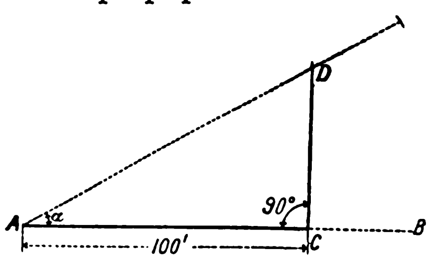

FIG. 70.—Setting out an Angle. Tangent Method.

three sides of this triangle are measured. At a suitable distance along the line AB set off the equal triangle KML. The line joining C and M will be parallel to the line AB.

Setting out a Given Angle.—With the aid of tables of trigonometrical functions of angles, any angle may be set out with the use of the chain alone or with the chain and optical square.

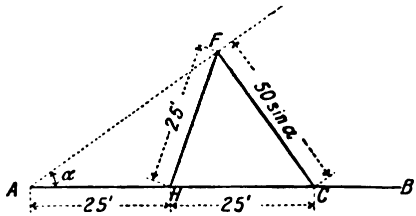

FIG. 71.—Setting out an Angle. Sine Method.

Tangent Method.—In Fig. 70 it is required to set off at point A a line making an angle α with the direction AB. Measure off a base AC any convenient round number of feet in length, say 100 ft. At C set off a perpendicular CD, having a length equal to 100 tan α. The line AD then makes the required angle with AB.

Sine Method.—In this method, as shown in Fig. 71, the base AC is bisected at point H, and the point F is determined, so that FH = HC and FC = AC sin α. The direction AF then makes the required angle with the direction AB.

Method by Sine of Half the Angle.—Fig. 72 illustrates this method. Point K is set out so that the distance AK is equal to the base AC and distance KC is equal to $2\text{AC} \sin \dfrac{\alpha}{2}$. Direction AK then makes the required angle with the direction AB. Of the three methods given this is the only one which will give passable results when the angle α approaches 90°. The other two methods should not be used for angles which are much over 60°.

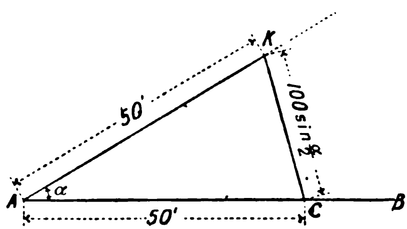

FIG. 72.—Setting out an Angle.

Overcoming Obstacles to Chaining.—In chain surveying a condition essential to the attainment of accuracy and expedition is that the chain lines should be so arranged clear of obstacles and obstructions that they can be lined out and measured directly without any

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trouble. As, however, the surveyor must be prepared to deal with any unavoidable obstacles that may occur, he should make himself familiar with the geometrical principles applicable to the solution of the following problems connected with the circumventing of hindrances in chain surveying:—

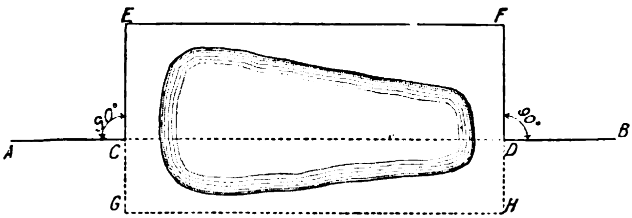

FIG. 73.—Chaining past Pond, &c.

Chaining past an Inaccessible Area.—Assuming that in this case there is no difficulty in ranging out the line on both sides of the obstruction the problem may be simply solved in the manner illustrated in Fig. 73. From points C and D on the chain line on either side of the obstruction perpendiculars CE and DF of equal length are set off to one side of the chain line, so that the line EF

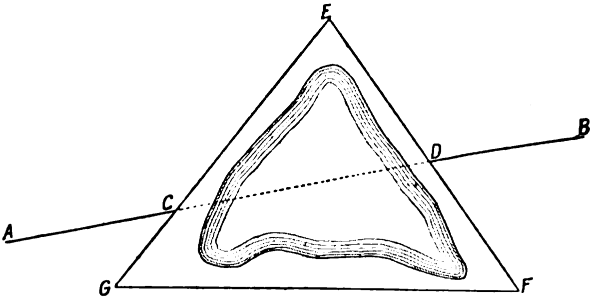

FIG. 74.—Chaining past and surveying Pond.

clears the obstacle. The measured length of EF is equal to the omitted distance CD on the chain line. The boundaries of the obstruction may be surveyed from the lines CE, EF, and FD, and from similar lines set out on the other side of it if required.

Another method which might be usefully applied where the boundaries of the obstructing area have to be surveyed, is illustrated in Fig. 74. The chain line is made to terminate on either

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side of the area in points C and D which lie on the sides of a triangle GEF conveniently arranged for surveying the area. The length of the omitted distance CD can readily be got by plotting the triangle GEF on a separate piece of paper and scaling the distance between

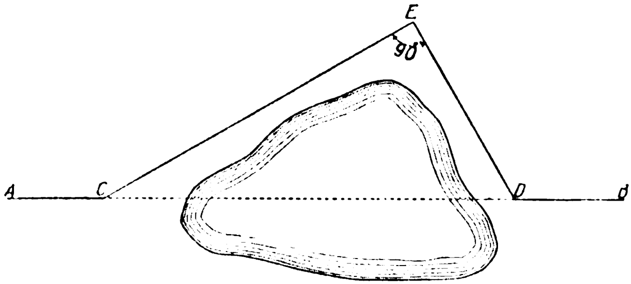

FIG. 75.—Chaining past Pond, &c.

the points C and D. The length of CD may also be calculated by the principles of trigonometry.

A simple method which might, under suitable circumstances, be applied is shown in Fig. 75. From a convenient point E lines

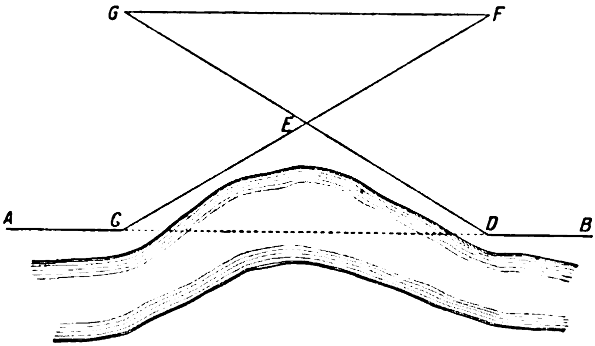

FIG. 76.—Chaining past Obstruction.

EC and ED are set off with the optical square or other instrument at right angles to each other so as to clear the obstruction, and meeting the chain line in points C and D. The lines CE and ED having been measured, the length of CD will be got from the equation $CD^2 = CE^2 + ED^2$.

Another method is shown in Fig. 76. From points C and D on

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the chain line lines CE and ED are set out and prolonged, EF being made equal to CE, and GE equal to ED. The length GF measured on the ground is then equal to the distance CD.

Chaining past an Obstruction which cannot be seen through.—If the conditions are such that the line can be correctly ranged out on

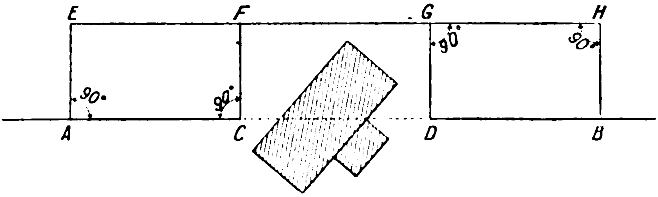

FIG. 77.—Prolonging Chain Line past Obstruction.

both sides of the obstacle, then any of the four last-mentioned methods may be applied to find the distance CD.

If, however, the line requires to be ranged out from one end and produced for some distance past the obstruction, one or other of the following methods might be applicable :—

Method by Parallel Line.—The chain-line having been ranged out from the left-hand end up to point C (Fig. 77), set off equal perpendiculars AE and CF a considerable distance apart. Range out the straight line EFGH past the obstruction and again set off two perpendiculars GD and HB each equal to CF. Points D and B are in line with points A and C, and may be utilised for prolonging the survey line towards the right. The measured length of FG is equal to the omitted distance CD.

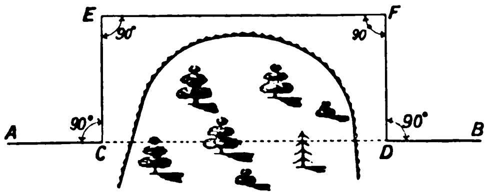

FIG. 78.—Prolonging Chain Line past Obstruction.

Method by Four Right Angles.—The optical square may be used to lay out successively the lines CE, EF, FD, DB, as shown in Fig. 78. Then if FD be made equal to CE, the line DB will lie on the prolongation of the line AC and the distance EF will be equal

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to the distance CD. This is rather a rough and ready method especially as regards the correct ranging out of the line AC beyond the obstruction.

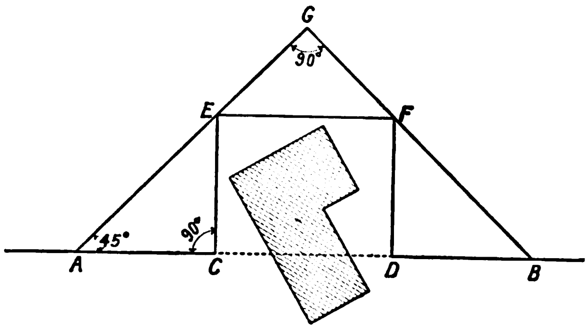

FIG. 79.—Prolonging Chain Line past Obstruction.

Method by Similar Triangles.—In Fig. 79, CE is set off equal to and perpendicular to AC. AE is ranged out to G and GB is set out perpendicular to AG, GF being made equal to EG, and FB equal

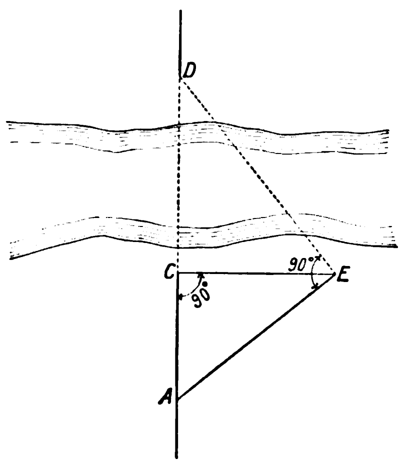

FIG. 80.

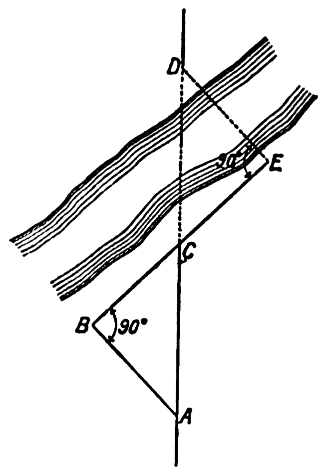

FIG. 81.

Distance across a River.

to AE. Point D is then determined by the intersecting distances FD and DB, which are each made equal to CE. Points D and B lie on the survey line and the measured length of EF is equal to the length of CD.

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A somewhat similar method can be devised, using equilateral triangles. The methods by triangles are not very satisfactory owing to the unavoidable shortness of the sides.

Distance across a River.—The distance across a river which is more than one chain length in width, or where the chain cannot be passed across, might be determined by the method shown in Fig. 80.

Points C and D are fixed on the survey line on either side of the river. From C a line CE is set out perpendicular to the survey line and of a length preferably about equal to CD. From point E a line EA is set out with the optical square or other instrument perpendicular to the line ED, and point A is marked where it meets the survey line. The distance AC is measured. The required distance CD is then equal to $\frac{CE^2}{CA}$.

Another method of using right-angled triangles is shown in Fig. 81. From a suitable point E the line EC is set out at right angles to the line ED to meet the survey line in point C. Line EC is then produced to B, BC being made equal to CE, and line BA is set out at right angles to BE, meeting the survey line in point A. The triangles ABC and CED are similar and equal, and the length of AC is, therefore, equal to the length of CD.

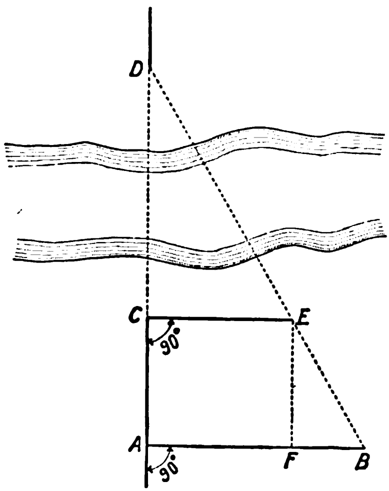

Fig. 82.—Distance across a River.

Fig. 82 shows still another simple device, by means of which the distance CD may be deduced. The line CE is set out at right angles to the survey line. From another point A the line AB is set out at right angles to the survey line, point B being at the same time put into line with points E and D. If AF is equal to CE the triangles BFE and ECD are similar, so that :—

$$\frac{CD}{CE} = \frac{EF}{FB}, \text{ or } CD = \frac{CE \times EF}{FB}.$$

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As the distances to be measured on the ground are AB, AC and CE, the above formula becomes :—

$$CD = \frac{CE \times AC}{AB - CE}.$$

Surveying Far Side of a River.—The principle of the above method can, under suitable circumstances, be applied to surveying the far side of a river without crossing it. A conspicuous mark, such as a boulder, plant, tree trunk, &c., is noted at each salient point of the bank, such as A, B and C in the Fig. 83. Using the optical square, points a, b, c are found where perpendiculars from A, B and C meet the survey line. To find distance aA, a convenient point D

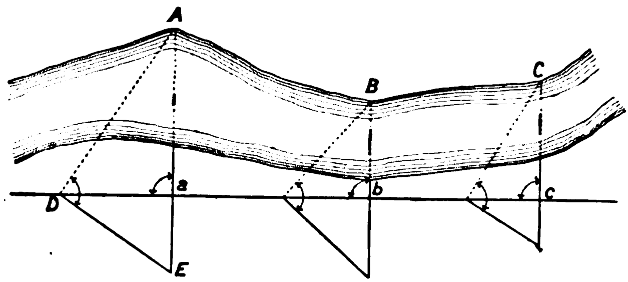

FIG. 83.—Surveying far Side of River.

is chosen on the chain line and DE is set out perpendicular to DA, point E being at the same time put into line with a and A. The length of aA is then equal to $\frac{Da^2}{Ea}$.

Surveying Pond, Wood, &c.—A pond or a thick wood through which lines cannot be sighted must be surveyed by lines run round the outside. Wherever practicable, one large triangle should be set out enclosing the area, as indicated in Fig. 84, and such other subsidiary lines should be run between the sides of the main triangle as are required for picking up the boundaries. In plotting the work the main triangle is first laid down. The subsidiary lines are then joined up and serve as checks on the accuracy of the work.

Fig. 85 illustrates the kind of arrangement which may be adopted where it is impracticable to lay out one encompassing triangle. A polygon ABCD is laid out enclosing the area. To enable the sides

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of the polygon to be plotted in correct relationship to each other some of the adjacent sides must be connected by ties or triangles. In the figure the relationship of the two sides meeting at A is fixed by prolonging those sides and forming the triangle A*fe*. The relationship of the sides meeting at B is fixed by the tie *gh*, which also serves as a survey line. Triangles or ties at the angles C and D are not required for plotting purposes, but are desirable as checks on the accuracy of the work. The lines might be plotted by first setting off the base *f*A*g*B and then plotting the triangles *f*A*e* and *gh*B. *e*A and B*h* produced, give the directions of the sides AD and BC respectively. The lengths of these sides being then laid off to scale, points D and C are fixed, and the scaled distance DC should be equal to the

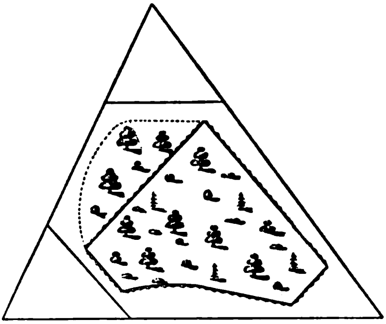

FIG. 84.—Surveying Wood, &c.

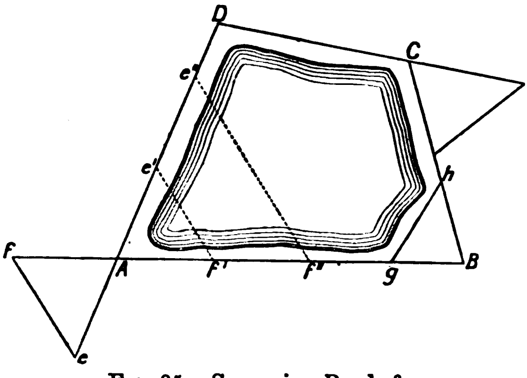

FIG. 85.—Surveying Pond, &c.

distance measured on the ground. The triangle at C affords a further check on the accuracy of the work.

It is evident that the triangle A*fe* might with advantage be plotted in the position A*f′e′*, and that less error would be introduced in plotting if it were laid off considerably magnified. That is, instead of plotting the triangle A*f′e′* to the actual measured dimensions, plot it to some multiple of those dimensions as $Af''e''$, and similarly with the other small triangles.

Chain Surveying in Towns.—In towns the survey lines must necessarily for the most part run along the streets, and their arrangement into a system of triangles is impossible. Chain surveying is, there-

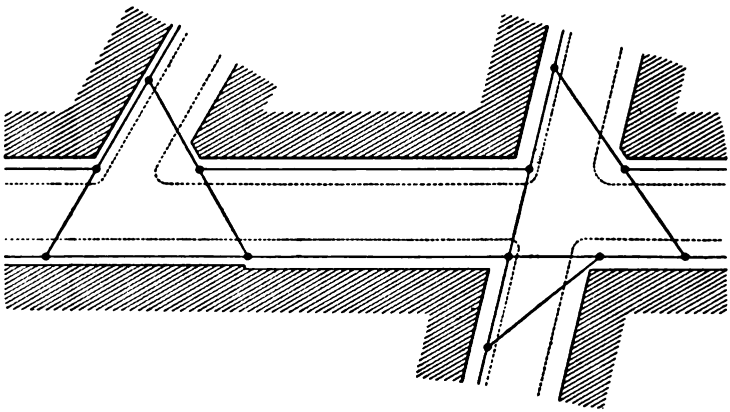

FIG. 86.—Chain Surveying in Towns.

fore, not adapted for town work. A length of street might, however, be quite well taken up with the chain alone along with short lengths of the branch streets. The branch survey lines are connected to the main survey line by triangles at the junctions of the streets, and as the accuracy of the work depends mainly on these triangles they must be made as large as practicable. Fig. 86 illustrates the sort of arrangement which may be adopted.

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CHAPTER VIII

PLOTTING THE PLAN

This chapter deals with the drawing instruments used in the plotting of survey plans and with the methods of procedure applicable to the commencing of the plan and its arrangement on the sheet, the laying off of the base line, the plotting of the main triangles, and the plotting of offsets, ties and details. Consideration is given to the errors which are apt to arise in plotting, and hints are given as to the proper methods of pencilling, inking-in, and erasing lines. Conventional methods of representing various features, such as fences and boundaries, roads, railways, buildings and various kinds of land, are considered and illustrated, and the matters of printing and lettering, colouring and tinting are also dealt with.

Paper.—For plans intended to remain as a permanent and accurate record of the work surveyed good, tough, seasoned hand-made paper should be used. The surface should be moderately rough and the quality such that ordinary rubbing and erasures can be made without spoiling the paper for taking colour and ink lines. Whatman's hand-made paper, double elephant size, is suitable for single plans, which should preferably be kept flat in a drawer. When required to stand much handling the paper should be mounted on holland. For large plans several sheets of paper may be mounted together on cloth. Plans which extend over a long narrow area, such as the general working plan of a railway, are often made on a continuous roll formed of single sheets mounted together on cloth. Rolled plans do not remain so true to scale as plans which are kept flat.

Pencils.—A good quality pencil, hardness 3H or 4H, should be used for plotting. For marking points and distances a hard pencil with a very fine round point may be used, but it is preferable to

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employ a pricker with needle point for this purpose. For drawing lines a fine chisel point should be used. The pencil should be held nearly vertical, its top being just slightly inclined forward in the direction of motion, and the flat side of the point held close against the straight-edge or ruler. For a right-handed person the directions of motion in drawing lines should be from the left-hand side of the paper towards the right-hand side and from the bottom towards the top.

Pricker.—A serviceable pricker may be made by fixing a portion of the point end of a sewing needle into the end of a pencil. The leg of a pair of dividers or compasses, if furnished with a good point, will often serve the purpose. Points marked off with a pricker may be rendered conspicuous, so as to be readily recoverable when wanted, by drawing a small pencil circle round them.

Scales.—Surveys made with the 66-ft. chain are generally plotted to one of the following scales: 10, 20, 30, 40, 50, 60, or 80 links to 1 in., or to some round multiple of one of those scales. If distances are required to be scaled in feet on a plan plotted to one of the above link scales, either a specially manufactured scale must be used, or a scale of feet corresponding to the link scale must be drawn. A scale of 66 ft. to 1 in. corresponds to the scale of 100 links to the inch. The foot scales corresponding to the ordinary link scales will, therefore, always contain an awkward number of feet per inch, namely, some multiple or sub-division of 66.

Surveys for engineering purposes are usually made with the 100-ft. chain or steel band, and plotted to one of the following scales or some round multiple of one of them: 10, 20, 30, 40, 50, 60, or 80 ft. to 1 in. In connection with railway work in Britain, survey plans are commonly plotted in feet to the scales 1 in. = 33 ft., and 1 in. = 66 ft.

The scales 4, 8, 16, 32, and 64 ft. to the inch are much used by architects. On plans drawn to these scales distances can be measured roughly with an ordinary foot-rule.

The principal scales of the Ordnance Survey maps of Britain are the following: the $\frac{1}{500}$, $\frac{1}{2500}$, and 6 in. to a mile or $\frac{1}{10560}$. The $\frac{1}{500}$ is known as the large Ordnance Scale and is used for the survey plans of towns. Maps of all cultivated portions of the country are

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prepared to the $\frac{1}{2500}$ scale, which is equivalent to 208⅓ ft. to the inch, and maps of the whole country are prepared on the 6 ins. to mile scale.

In plotting the plans flat boxwood rules about 12 ins. long, with both edges bevelled and a different scale engraved on each are most commonly used. The divisions are figured decimally. Scales whose divisions on one edge are intended to represent links may have the other edge divided into feet to correspond. A common form of scale, as shown in Fig. 99, has 10 divisions to the inch along one edge and 20 along the other, other similar combinations being 30 and 40 divisions to the inch and 50 and 60 divisions to the inch.

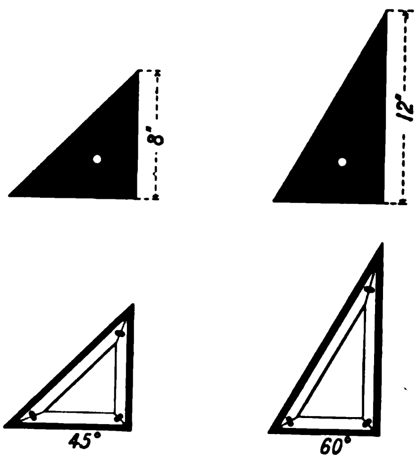

Fig. 87.—Set-squares.

The main divisions are numbered from both ends so that distances may be scaled and marked off in both directions. The scale marked 10 may be used to plot work to the scale of 1 in. to 1 ft. In that case the numbers along the scale will be read as feet, and each small division will represent $\frac{1}{10}$ of a foot. Or it may be used to plot to the scales of 10 ft., 100 ft. or 1,000 ft. to the inch. In the same way the scale marked 20 can be used for plotting to 2, 20, 200, &c., feet to the inch.

Set-squares.—The draughtsman should have two good-sized triangular set-squares for drawing perpendicular and parallel lines and for general use as straight-edges in drawing and inking-in short lines. Fig. 87 shows common forms of set-squares and sizes suitable for general use. Triangular framed set-squares of pear wood or mahogany with ebony edge are reliable and cleanly in use. Set-squares formed of triangular sheets of vulcanite or transparent celluloid are now common. They pick up dirt and soil the paper much more readily than wooden set-squares and should always be

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rubbed clean before use. It is essential that the set-squares used in plotting survey plans should be accurate, especially as regards the right angle. To test if this is correct apply the base of the set-square to a straight-edge on a sheet of paper, as shown in Fig. 88, draw a pencil line along the perpendicular side, and reverse the set-square. If the edge coincides with the pencil line the right angle is correct.

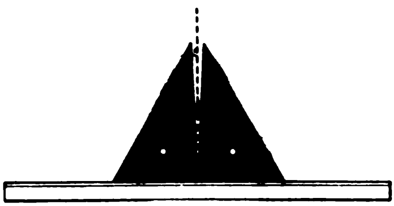

FIG. 88.—Testing Set-square.

Straight-edge.—All instruments employed for drawing accurate straight lines must have their edges perfectly true and straight. The accuracy of a drawing edge may be tested in the manner illustrated in Fig. 89, by drawing a pencil line and then reversing the straight-edge, end for end.

Long straight-edges are most reliable if made of steel. A suitable size for drawing long base lines, &c., is 6 ft. in length and about 2·5 ins. by 0·1 in. in cross-section with one edge bevelled.

If accurate straight lines are required longer than the length of the available straight-edge, a fine thread should be stretched tight between the ends of the line and a few points along its length

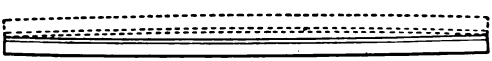

FIG. 89.—Testing Straight-edge.

transferred accurately to the paper by means of a pricker. The straight-edge should then be used to join up the line through these points.

Compasses.—The various instruments used in drawing circular arcs are spring bows, bow compasses with or without lengthening bar, beam compasses, and manufactured curves. These are illustrated in Figs. 90 to 95.

Spring bows can be accurately adjusted by the thumb screw to the exact radius required, and are used for drawing small circles within about an inch radius. Bow compasses with jointed legs

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are used for circles up to about 5 ins. radius, and with lengthening bar up to about 9 ins. radius. In using the compasses the legs should be bent at the joint so as to bring the points perpendicular to the paper, this being specially necessary in the case of the drawing pen point. Compasses used with a long lengthening bar are apt to be deficient in steadiness.

In the beam compasses, as shown in Fig. 94, the pivot point and the drawing point form detached portions of the instrument and can be clamped at any required distance apart on a beam of wood which, for the sake of stiffness, is usually made of T section. The drawing point is furnished with a tangent screw by means of which the final adjustment to the exact radius is made. To set the beam compasses to the required radius, first mark off by scale the length of the radius along a pencil line drawn, say, near the edge of the sheet of paper. Then, having set the points approximately to this length, apply the pivot point to one end mark and turn the tangent screw until the drawing point coincides with the mark at the other extremity of the length. The pivot point is then set to its centre and the arc is struck.

For drawing accurate pencil circles of large radius a narrow strip of tough seasoned drawing paper serves excellently in place of the beam compasses. The required radius is marked off along a pencil line drawn down the centre of the strip, and fine holes are pricked through the end points. A pricker or common pin passed through one of these holes forms the pivot, and a fine pencil point passed through the other serves to draw the arc. Just sufficient tension should be maintained on the paper strip to keep it taut.

Manufactured Curves, formed of flat strips of pearwood, cardboard or vulcanite, and having both the convex and concave edges turned to the same radius, can be obtained in sets having radii varying from 1½ ins. to 240 ins. or more. These are useful for drawing and inking-in curved lines, and are indispensable for railway work.

Dividers.—The dividers, similar to the bow compasses, but with both legs stiff and sharp-pointed, are not much required in plotting plans. Their chief function in surveying work is to measure distances on the finished plans. Short distances are measured by setting the dividers to the whole length and then applying them to

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the scale on the plan. Longer distances are measured by stepping. The dividers are set to a round distance on the scale, and then

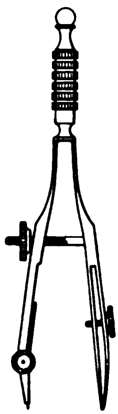

Fig. 90.—Pen Spring Bows.

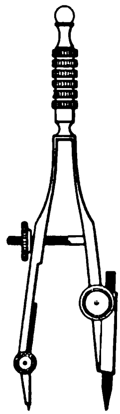

Fig. 91.—Pencil Spring Bows.

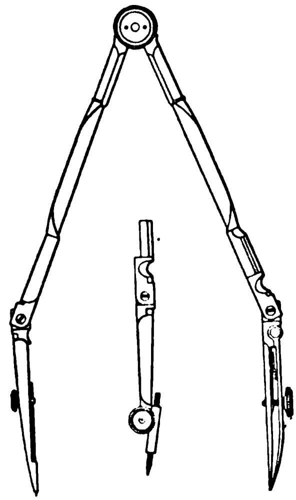

Fig. 92.—Pen and Pencil Bow Compasses.

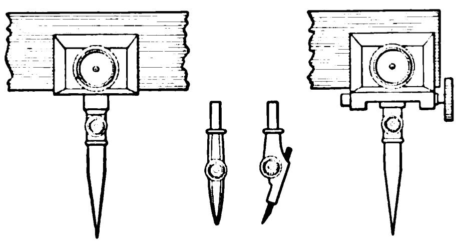

Fig. 94.—Beam Compasses.

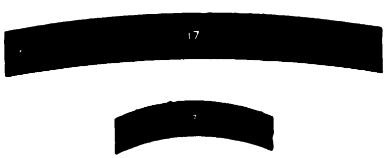

Fig. 95.—Manufactured Curves.

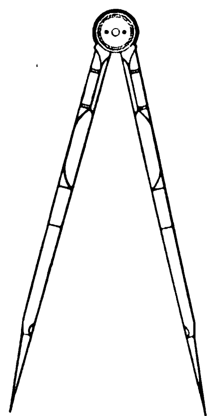

Fig. 93.—Dividers.

stepped along the line from one end, the equal distances thus measured off being added up mentally as the stepping proceeds. The last fractional interval is taken on the dividers, measured on

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the scale, and added to the total length of the equal steps to give the total distance between the points.

For use in the field the surveyor should be provided with a pair of pocket dividers, having a screw-on shield or other device to protect the points. Ordinary dividers may be carried in the pocket if their points are inserted in a piece of cork or indiarubber.

Parallel Ruler.—The ordinary T-square sliding on the edge of a drawing board is of little service in plotting surveys. A long, heavy, parallel ruler is more generally useful. The instrument, which should be at least 2 ft. long, consists of a heavy bar of brass or electrum about $2\frac{3}{4}$ ins. wide, with parallel bevelled edges and mounted on a pair of rollers (Fig. 96). The rollers are of equal diameter and are rigidly connected to the same axle so that they roll together and travel over equal distances. The ruler, therefore, keeps a parallel direction as it travels across the paper. To test

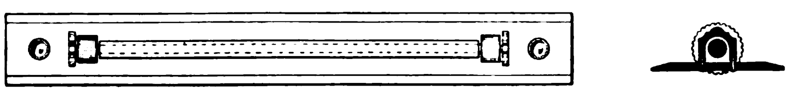

Fig. 96.—Parallel Ruler.

the accuracy of a parallel ruler, draw a pencil line along its edge on a sheet of paper, then roll the instrument a considerable distance across the paper and draw another line along the same edge. Reverse the ruler end for end, apply the same edge to the first line, roll the instrument across the paper and see if the edge now coincides with the second drawn pencil line. If it does the ruler is correct.

With the use of the parallel ruler and set-squares parallel lines and perpendicular lines can be drawn in any direction on the paper.

Drawing Pen.—The drawing pen is illustrated in Fig. 97. The nibs should be of good quality tempered steel, otherwise they soon become blunt and sharp uniform lines cannot then be drawn. The pen is more easily cleaned, sharpened, and adjusted if one of the nibs is hinged so as to open wide when the screw is taken out. To get good lines the nibs must be sharp, of the same length and shape, and exactly opposite one another. In drawing ink lines the pen should be held and handled in a similar manner to the pencil, with

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the screw head away from the edge of the ruler. Uniform firm pressure should be exerted downwards on the paper, and only a very slight constant pressure against the straight-edge, so as to ensure lines of even thickness. Ink should not be allowed to dry between the nibs. They should be cleaned out thoroughly and often by drawing the fold of a duster between them without altering the screw.

Commencing the Plan.—The scale of the plan will be fixed from considerations of the purpose for which the plan is intended, and the size of the sheet of paper required will be governed by the scale chosen. Having determined the necessary size of the sheet, lay it down flat on a table or board, previously rubbed clean, and fasten the edges down with cloth- or leather-covered weights, or with drawing pins. See that the instruments are clean. Keep covered over with paper any portions of the plan that are not being worked upon.

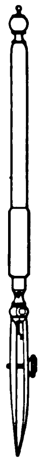

Fig. 97.—Drawing Pen.

Arrangement of Plan on the Sheet.—If an existing plan of the area is available, even though to a considerably smaller scale, a proper arrangement of the plan on the paper can be readily arrived at. Lay off on this plan the position of one of the main survey lines near the centre of the work, and on a piece of tracing paper draw a rectangle representing the size of the sheet on which the work is to be plotted, reduced to the scale of the existing plan. Place the tracing paper over the area and twist it about until the boundaries of the area appear in good position, relative to the rectangle, keeping in view the provision of space for the title, scale, north point, notes, &c., and the effect these will have on the symmetry of the completed plan. Having fixed the rectangle about the area, mark on it the position of the main survey line and its end points, and produce this line both ways to meet the edge of the rectangle. Reproduce this line and the survey line in accurate relative position on the sheet of drawing paper. This survey line so fixed will serve as a base on which to construct the system of triangles, and the work when plotted will lie in its predetermined position on the paper.

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Where there is no existing plan available the main triangles should be roughly plotted to the scale of the plan and the limits of the area sketched round them. The tracing paper can then be applied direct over the sheet of drawing paper and, when the area is judged to be in proper position, one of the main survey lines may be pricked through, measured off accurately to length and used as a base for the system of triangles.

Laying off the Base Line.—A reliable straight-edge must be used for laying off the base line and for drawing all survey lines. If any line is very long a stretched thread should be used in the manner already explained. If it will not be objectionable on the finished plan, the base and survey lines may be inked in with thin faint lines of blue or red colour.

A fine pricker will be used for marking off the lengths of lines. To ensure accurate work the eye should be placed opposite the

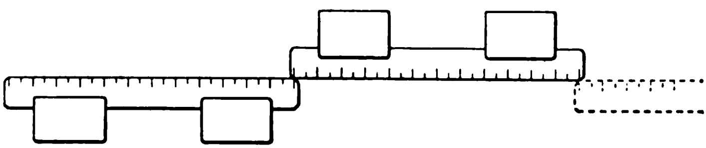

FIG. 98.—Laying off long Distance.

division of the scale to be pricked off, and the pricker should be held vertical. Long distances will be marked off in whole scale lengths. or the nearest round figure if the scale ends at an odd distance. The most accurate method of laying off long distances is by using two scales in the manner indicated in Fig. 98. The scales are placed one on each side of the line, their end divisions are made to coincide, and they are alternately moved forward and fixed down by weights.

Plotting the Main Triangles.—As a preliminary to the plotting of the main triangles the surveyor should prepare a sketch of the system of survey lines with all stations numbered, and with the lengths of all sides of triangles and other survey lines plainly figured. By keeping this before him during plotting he will save much turning up and searching of the notebook.

The triangles which are directly connected to the main base line will be plotted first. To plot the third station of a triangle,

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from the ends of the base as centres sweep intersecting arcs with the lengths of the sides as radii. The intersection of the arcs fixes the position of the station. Where the lengths of the sides are within the length of the scale it is quicker to draw only the first arc for each triangle and to use the scale itself as radius for finding the intersection instead of the second arc. Before permanently marking a station found thus by intersections test whether both sides of the triangle scale correctly. If there is a slight error adjust the point to true position. Then test its accuracy further by any proof lines which may have been taken. If the result is satisfactory, make a definite prick mark at the point and draw in the sides of the triangle. Proceed similarly with the rest of the triangles attached to the base line and then with those more remote. No plotting of detail should be commenced until the whole system of triangles has been completed and checked.

Plotting Offsets.—The plotting of offsets and ties will follow roughly the order in which they were measured in the field. Where few offsets occur on a survey line the quickest method will be to first mark off along the line the positions of all offset points, and then erect perpendiculars at these points and prick off the lengths of the offsets to scale. Offsets which have been set out at right angles by the eye in the field may be plotted by placing the scale at right angles by estimation across the survey line. Perpendicularity of the longer offsets should be ensured by using the right-angle set-square.

Where there are many offsets the method of plotting illustrated in Fig. 99 is recommended. A short sliding offset scale is used, having the zero of graduation at its middle and with an index mark on the sliding base. The scales are set ready for use by placing the offset scale at right angles to a survey line with its zero at the commencing station and then setting and fixing the ordinary scale parallel to the survey line and with its zero at the index of the offset scale. On sliding the offset scale its zero should travel along the survey line. Then to plot, say, an offset of 27 ft. at distance 253 on the survey line, set the index to 253 on the ordinary scale and make a prick mark at division 27 on the offset scale. By this method of plotting, points do not require to be marked off along the survey line, and the drawing of pencil perpendicular lines is avoided. If

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a continuous boundary is being plotted the points should be joined up freehand as they are plotted. Isolated points which are not immediately required should be marked with a pencil circle for the sake of easy recovery.

Plotting Ties.—Ties are plotted by intersecting arcs drawn with the spring-bow compasses. Where the offset scale is being used to plot perpendicular offsets it may also be used to mark off the centres for the ties along the survey line. Wherever check measurements have been made they should be applied to test the accuracy of the work as the plotting proceeds.

Plotting Details.—Details of houses and other objects which have been separately sketched should be plotted concurrently with the

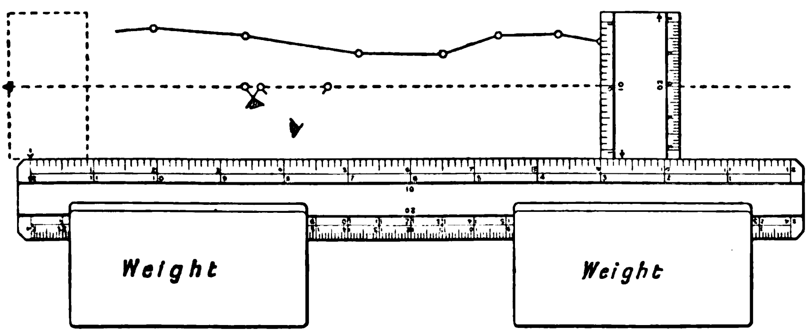

FIG. 99.—Plotting Offsets, &c.

survey lines. Errors are more readily avoided and sooner detected if all the plotting is completed as the plan progresses.

Errors in Plotting.—The error in measuring off a distance with an engine-divided scale should be very small. It should be within $\frac{1}{100}$ in. in any single measurement, and should not much exceed that amount in the length of a double elephant sheet. The serious errors in plotting arise principally from misreading the notebook, blunders in reading the scale, measuring from the wrong point, drawing lines between the wrong points. The notebook should be so placed in front of the draughtsman that figures and writing can be read in their natural position. Errors in plotting due to misreading the scale usually amount to an even round number of feet. An error of 10 ft. is probably most common, and is most liable to

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occur when the scale is figured only at alternate divisions, namely, the even tens. Mistakes of 1 ft. and 100 ft. also occur. Large mistakes of this class will generally be detected. With scales whose divisions are figured from both ends the mistake may be made of looking at the number on the wrong row, and hence marking off the wrong distance. This mistake is most liable to occur at distances near the centre of the scale. Errors due to measuring from the wrong point, such as laying off the wrong offset from a point on a survey line, or errors due to drawing lines between the wrong prick marks can be avoided only by exercising great care and constantly applying checks. Serious errors are, as a rule, oftener introduced in the plotting than in the field work, and the employment of systematic methods of procedure in plotting and checking is essential to the attainment of an accurate plan.

Pencilling.—The pencilling of the work should be done in fine, firm lines. Do not press so hard on the point as to indent the paper or render the lines difficult to erase. Particular care must be taken to see that the proper prick marks are joined up. In pencilling boundary lines pay attention to the definite angles and see that the lines are drawn through all the plotted points. So far as possible leave the prick marks visible so that the ink lines may be drawn exactly through them. In plans of single sheet size complete the pencilling and check it carefully before starting to draw the lines in ink. With large plans or long rolled plans it may be advisable to do the inking-in in sections as the plotting proceeds, as pencil lines soon become blurred and indistinct when a plan is subjected to much handling.

Inking-in.—Chinese ink freshly rubbed down from the stick gives best results. See that the porcelain dish is clean before rubbing down, and keep the ink covered to prevent the access of dirt and retard evaporation. Make the ink thick enough to give a dense black line, but not so thick that it clogs the pen. The lines should not be drawn too narrow. They should be wide enough and firm enough to remain distinct after a plan has become rubbed and dirtied by much use. If satisfactory permanent ink lines are desired, it is essential that the drawing pen should be sharp and pressed firmly into the paper. Lines drawn with a blunt pen are easily rubbed off. If a line as first drawn appears ragged or

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contains gaps owing to the paper being greasy in places, go over the line again in the same direction with the drawing pen, taking care not to make it too thick. As a general rule, it is desirable to commence inking at the top left-hand corner of the paper and to work downwards and to the right, and curved lines, especially if drawn with a compass, should be inked in before the adjoining straight lines, the positions of the tangent points having previously been carefully marked in pencil. Where, however, manufactured curves are used for ruling in the curved lines, as in drawing railway lines, &c., it is better to work continuously in one direction (left to right), taking the straights and curves in order. In pencilling railway curves and such like on a plan, jot down the radius of curve used and mark its tangent points. Draw the ink lines accurately over the prick marks so as to hide them if possible, see that the lines meet exactly at angles, without overlapping or showing any gap, and take precautions to avoid the sleeve, ruler, or set-square rubbing over the wet lines. Keep the plan as clean as possible, by covering up the portions that are not being worked upon, and see that there is no dust or eraser rubbings in the path of the drawing pen.

Erasing.—Erasures of ink lines are difficult to effect and always leave their mark on the surface of the paper. The use of a knife for erasing lines usually results in spoiling the surface. Good ink-eraser—a hard rubber mixed with grit—applied gently and with patience is best for the purpose. When the line has been erased brush off the dust and grit, rub the erasure over with soft india-rubber to remove any grit sticking to the paper, clean away the effects of this rubbing, and then polish the surface of the paper with some hard and smooth substance, such as the end of an ivory scale or the rounded end of an ivory drawing pen handle. Fine sandpaper is sometimes useful for erasing purposes where the ink lines have been well cut into the paper.

Conventional Signs.—Some of the conventional signs employed in representing various objects and features on survey plans are illustrated in Figs. 100 to 109.

Full Black Lines.—All definite and permanently marked boundaries and outlines of existing objects should in general be represented by full black lines.

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Dotted Lines.—Dotted lines are employed to represent boundaries which are indefinite and generally somewhat unimportant. Paths, unfenced roadways, edges of slopes, unfenced edges of woods, kerb lines of streets, and divisions between cultivated land and moorland are examples of features which are generally shown by dotted lines. Dotted lines appear neatest when drawn with very short uniform dashes, almost dots, closely and uniformly spaced.

Coloured Lines.—Coloured inks and water colours are not so permanent as Chinese black ink, and some colours fade rapidly. They should be used as sparingly as possible on plans intended to be kept as permanent records. Proposed works, proposed divisions of land, &c., are usually drawn in red lines. Railway lines are often drawn in blue.

Fences and Boundaries.—On small scale plans all sorts of definite boundaries, such as fences, hedges and walls, are usually represented by a single full black line. See Fig. 100.

On plans drawn to a fairly large scale, a dot-and-dash line is generally used to distinguish fences from other boundaries. A good appearance is obtained if the dashes are made about 0·15 in. long, uniformly spaced, and with intermediate dots which are merely round dots and not short dashes.

Walls are shown by double full lines when the scale is large enough to show the thickness.

Hedges are shown by single full lines with a conventional representation of bushes drawn over them.

Some conventional representations of gates are shown on the boundaries illustrated in Fig. 100.

Roads.—The methods of representing roads vary with the scale of the plan, and are generally as indicated in Fig. 101.

Railways.—The methods of representing railways vary also with the scale of the plan. Usual conventions to represent a single-line railway and a double-line railway where the scale is too small to permit of showing the actual width of the railway or details of cuttings and embankments are shown in Fig. 102, where also a portion of a double-line railway is shown, with the amount of detail appropriate to a scale of 1 in. to 80 ft. or larger.

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FENCES AND BOUNDARIES.

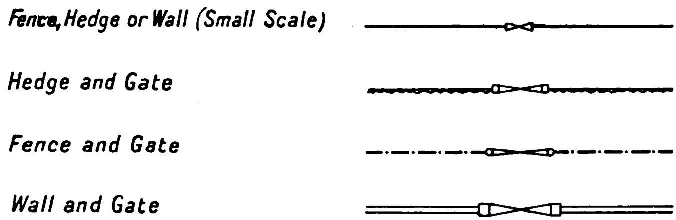

Fig 100.

ROADS.

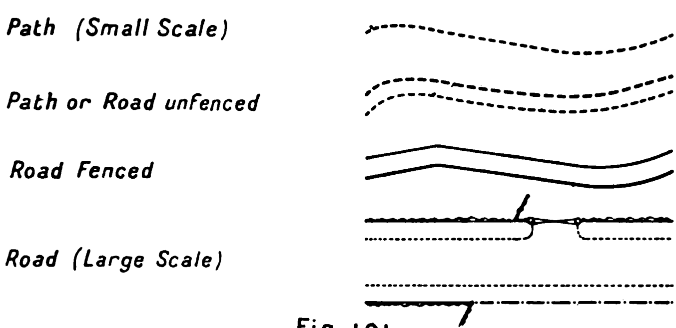

Fig 101.

RAILWAYS.

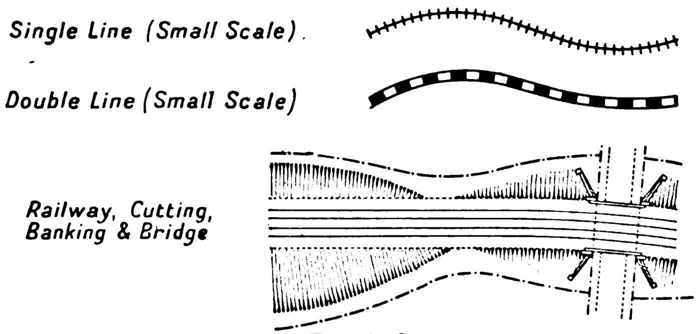

Fig. 102.

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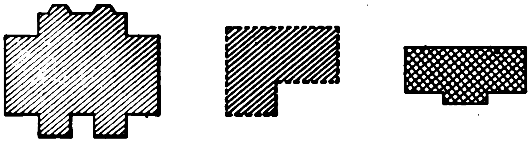
BUILDINGS.



House. Shed with open side. Glasshouse.

Fig. 103.

VARIOUS KINDS OF LAND.

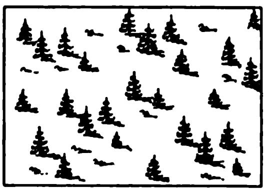


Fig. 104. Fir Wood

Fig 105. Mixed Wood.

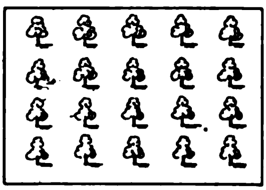


Fig. 106. Orchard.

Fig 107. Moorland

Fig. 108. Marsh and Pond

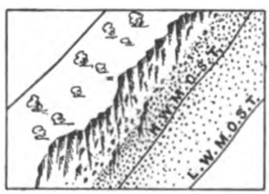
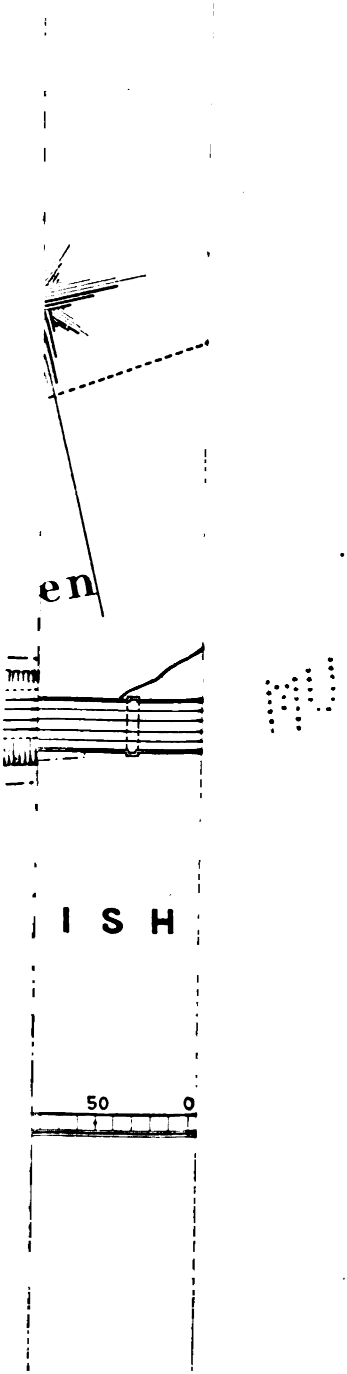


Fig. 109. Cliffs and Shore



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Buildings.—Methods of representing buildings are shown in Fig. 103. Sheds with open sides are shown dotted in outline. Glass-covered houses are generally shown cross-hatched in black or blue lines. Masonry buildings may be distinguished from wooden buildings by showing the former coloured in light Indian ink and the latter in light brown, but it is quite common to colour all buildings light Indian ink. An effective method of showing up buildings is to hatch them with thin parallel ink lines evenly spaced. This method is more laborious than colouring, but causes less distortion of the plan. The employment of shade lines helps to make the buildings stand out more graphically. To fix which lines should be emphasised by thickening, imagine that rays of light are coming from the top left-hand corner of the plan and crossing it diagonally at 45°. Those sides which the light strikes remain of ordinary width, the others, which would be in shadow, are thickened towards the inside of the building. Shade lines may, with good effect, be about three times as thick as the ordinary lines.

Various Kinds of Land.—Conventional representations of fir wood, mixed wood, orchard, moorland, marsh and pond, and rocky shore with high and low water marks are shown in Figs. 104 to 109.

Example from Survey Plan.—Plate I., which is copied from a portion of a survey plan of a double-line railway plotted to the scale of 1 in. to 66 ft., and reproduced here to a somewhat smaller scale, embodies a considerable number of conventional signs and gives a good idea of how they should be applied on a survey plan. On the survey plan from which the plate is copied the rails were shown in blue lines, the water of the reservoirs was shown by shading in blue colour round the edges, and the shade lines of the earth slopes were drawn in thin Indian ink, and were consequently less pronounced than the full black lines of the plate.

The plate also shows scale, north point, and specimens of lettering.

Printing and Lettering.—Plate II. shows the usual styles of lettering which may be employed in printing the necessary information on a survey plan.

The student who desires to become proficient at printing and lettering should start with the practice of the vertical block printing. His aim in the first place should be directed towards the attainment

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of correct form and proportion in making the letters, and he must not be satisfied till he can form the letters directly with the printing pen without any preliminary pencilling. The acquirement of a rapid, neat and effective style of finishing the letters will then come as the result of further and persistent practice.

When the student has mastered the vertical block printing he will find that the attainment of a neat hand in any of the other styles will be a comparatively easy matter.

The vertical block lettering, varied in size according to the importance of the particulars described, may be used throughout for the lettering of survey plans and, when neatly executed and arranged, is always effective and in good taste, but to most draughtsmen it is rather a laborious method. The small thick-and-thin italics can be rapidly executed and is usually very neat on paper plans, but is not very suitable for tracings which are to be used in making sun-print copies, as the thin portions of the letters hardly come out on the prints. For this reason the small sloping italics of uniform thickness is in much more common use nowadays. It is also probably the most rapid of all styles of hand printing.

In order to avoid an appearance of sameness over the plan, it is desirable that a limited number of variations of the character of the printing should be made in such a way as to emphasise the different character and importance of the information which it denotes. For example, vertical block letters, Roman capitals, and small Roman letters may be used for various classes of important information, while the general run of small printing is done in small italics. A study of the published plans of the Ordnance Survey of Britain will furnish useful ideas as to the size and character of printing appropriate to the various classes of information on the several scales.

The title of the plan should be formed, as a rule, of simple plain lettering in several lines, forming a well-balanced whole, in the form of an oval, if possible. Vertical printing has always a much better appearance in a title than sloping printing, and for a simple title there is probably nothing better than a combination of one or two lines of Roman capitals with one or two lines of vertical block, the size of the lettering in the several lines being varied with their importance. A thin ink line drawn under a line of printing will often help to make it stand out and appear clean and straight, but the overloading of a title with lines and scrolls should be avoided.

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ABCDEFGHIJKLMNOPQRSTUVWXYZ&

ABC abcdefghijklmnopqrstuvwxyz & RSTU

Vertical Block or Gothic Capitals and Smalls
and Open Block Capitals.

ABCDEFGHIJKLMNOPQRSTUVWXYZ&

abcdefghijklmnopqrstuvwxyz &

Roman Capitals and Smalls

ABCDEFGOPQRSW abcdefghijklmnopqrstuvwxyz

ABCDEFGPQRSW abcdefghijklmnopqrstuvwxyz

Sloping Block and Italics and Sloping Roman and Italics

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The title should generally be made to include :—

(*a*) The description or name of the area surveyed or the name of the estate, owner or principal for whom the work is done.

(*b*) General description of locality and purpose of survey.

(*c*) Particular description of the work shown on the plan, usually in the form " Plan showing proposed Road," or as the case may require.

The title should where possible be placed above the work on the vertical centre line of the sheet, but on survey plans of irregular areas the appearance of the plan may often be enhanced by suitably disposing the title in some vacant space between the work and the edge of the sheet. The title should never be placed within the work of the plan. Where the survey occupies a series of plans it will usually be preferable to make the title small and place it in the upper or lower right-hand corner along with the sheet number, so that when a number of plans are lying flat together in a drawer any particular one required can be found at once by turning up the corners.

Colouring and Tinting.—The application of large areas of colour to original copies of important survey plans is not desirable, as it causes alterations of the dimensions of the paper. On such plans where different areas have to be distinguished from one another it is better to confine the colour to an edging round the boundary of the area. For a small area the edging should be a single narrow band of fairly bright colour around the inside of the boundary. The brush should be held with its point towards the inner edge of the band and applied in short strokes towards the draughtsman. For large areas a broader edging consisting of a strip of light colour, ¼ in. to ½ in. wide, with a band of deeper colour about one-third as wide next the boundary may be used.

In tinting large areas the surface of the paper should first be cleaned, and all pencil lines, except those which are left to be afterwards inked in, should be erased, as otherwise they will be fixed by the colouring. The colour, palette, and water used should be clean, and sufficient tint should be mixed down to cover the whole area at one application. The sheet of paper should be fastened down to a drawing board, the back of which should be raised so as to give the paper a slope towards the draughtsman. A large brush

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should be used, and the tinting should commence from the top left-hand portion of the area and be continued simultaneously towards the right and downwards. The wet brush is applied from left to right along the upper portion of the boundary and then, with rapid motions of the brush, the tinted area is extended downwards. The colour drains towards the bottom edge of the wetted portion, and the whole art of tinting consists in constantly applying just so much colour as will keep the wetted surface continually draining downwards without allowing any large quantity to collect and run off in a trickle, and in executing the work so rapidly that no portion of the colour along the lower edge has time to dry before it is led further down. When the lower boundary is approached the brush is used somewhat dry, being freed from colour as required, and is employed to collect the surplus tint and complete the colouring to the edge of the area, working again from left to right.

Where overlapping areas have to be distinguished or where an area included within a larger coloured area has to be marked for separate recognition it is better to use cross-hatching with coloured lines for the purpose, rather than to superimpose one tint on the top of another of a different colour.

Date and Particulars of Survey.—An original survey plan should always have a note giving the date when the work was executed and should be signed by the surveyor in charge. If a plan is partly compiled from other plans the note should give particulars as to the extent plotted from actual survey, and the extent and source of the compiled portion.

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CHAPTER IX

COMPASS AND SEXTANT SURVEYING

THE instruments principally used by the surveyor for the measurement of angles are the surveyor's compass, the prismatic compass, the box sextant, and the theodolite. The three first-mentioned instruments are used for measuring horizontal angles (the sextant can, however, measure angles in any plane), and are light, easily portable, and rapid in use, and give results to a useful degree of accuracy for certain purposes. The theodolite may vary from a light instrument easily carried by one person up to a heavy instrument requiring special arrangements for its transport, and may have widely varying degrees of precision, but contrasted generally with the other three instruments, it is much heavier, much more awkward to carry, and requires a considerable expenditure of time and care in order to set it up on the ground in readiness for reading an angle. On the other hand, the theodolite gives much more accurate results than the other instruments, and can also be used for measuring vertical angles.

This chapter will deal mainly with the principles, construction, and use of the surveyor's compass, prismatic compass, and box sextant, and their employment in surveying.

The Magnetic Needle.—The earth acts as an immense magnet and exerts a directive influence on a magnetised bar of cast steel. Suppose that a bar of steel symmetrical about its centre is not magnetised and is suspended at its centre of gravity in such a way as to be free to rotate both vertically and horizontally and so that when pointed in any direction it does not tend to move away from that direction. Let the bar now be magnetised and again suspended as before. It will now oscillate about the point of suspension and will gradually settle into a definite position which will not be in a horizontal plane. The bar will be pointing in a

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slanting direction towards the ground. By suitably weighting the high end the bar may be brought to a horizontal position, and it will then have a definite direction in the horizontal plane, and if disturbed it will oscillate about and finally settle back into the same position. The horizontal angle which the direction of the bar now makes with the direction of true north is the "magnetic declination" at the place, and the vertical angle through which the bar required to be rotated to bring it from its original slanting position into the horizontal position is the "magnetic dip" at the place.

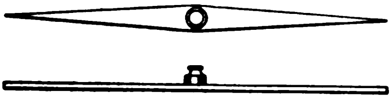

FIG. 110.—Broad Needle.

The two most common forms of the magnetic needle used in surveying instruments are the "broad needle," illustrated in Fig. 110, and the "edge bar needle," shown in Fig. 111. The needles are bored in the centre and fitted with a cap containing a hollow coned bearing of agate or other very hard stone. The needle swings on a pointed pivot of hardened steel. A lever arrangement is usually provided for lifting the needle off its bearing when not in use, so as to prevent unnecessary wear of the bearing with consequent increase of friction. The tendency to dip is usually counteracted by placing a small sliding weight on one arm of the needle. As the magnetic dip is variable over the earth's surface the weight may require shifting for different localities in order to obtain horizontal balancing of the needle.

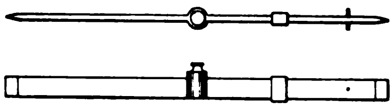

FIG. 111.—Edge Bar Needle.

Magnetic North and Magnetic Meridian.—A freely swinging magnetic needle properly balanced and centred and uninfluenced by sources of local attraction will come to rest in a direction which for most parts of the earth's surface is not widely different from true north and south. The end which points in the northerly direction is called the "north" end, or better, the "north-seeking" end of the needle. The direction in which this end of the needle points at a given place and time is the "magnetic north" for that place and time. The "magnetic meridian" of a place at a given time means the magnetic north and south direction at that time.

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Magnetic Declination.—As already defined, magnetic declination at a place at a given time is the horizontal angle which the magnetic needle makes with true north at that time, or it is the horizontal angle which the magnetic meridian makes with the geographical meridian. The declination at some places on the earth's surface lies to the west of true north, while at other places it lies to the east of true north. In Britain, in 1915, the declination was westerly throughout, varying from a maximum of about 22° on the west coast of Ireland to a minimum of under 15° at Yarmouth and Dover. Lines joining places of equal declination, known as isogonic lines, tend generally in a N.N.E. direction in Britain.

For surveying purposes the direction indicated by the magnetic needle can be considered as constant only over a limited area. In passing from Yarmouth in a W.N.W. direction to the west coast of Ireland the direction of the needle changes on an average 1° for every sixty statute miles, or at the average rate of one minute per mile.

The magnetic meridian at a given place is not constant, but varies continuously with time, and the magnetic declination alters correspondingly. The main variations of declination are (1) secular variations, (2) diurnal variations, (3) irregular variations. The secular variation is a gradual and continuous alteration of the magnetic declination which goes on from year to year with varying rate. The variation at London since 1580 is shown graphically on the following diagram (Fig. 112). The westerly declination of the needle is at present decreasing at all places in Britain. The British Admiralty publish a chart of the world showing lines of equal magnetic declination.

The diurnal variation is a daily oscillation of the needle to the extent of a few minutes, usually less than six, on either side of its mean position. The daily oscillation is not constant in amount. It is greater in summer than in winter, greater in high latitudes than at places near the equator, and is variable from year to year.

Irregular variations of large amount up to 1° or 2° sometimes occur suddenly, and are known as magnetic storms.

Local Attraction.—The magnetic needle may be attracted and prevented from indicating the true magnetic meridian when it is in proximity to magnetite and certain eruptive rocks in the earth,

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or to masses of steel or iron, such as bridges, rails, steel structures, water pipes, &c., or to electric cables or wires transmitting currents. Local attraction denotes any influence, such as the above, which

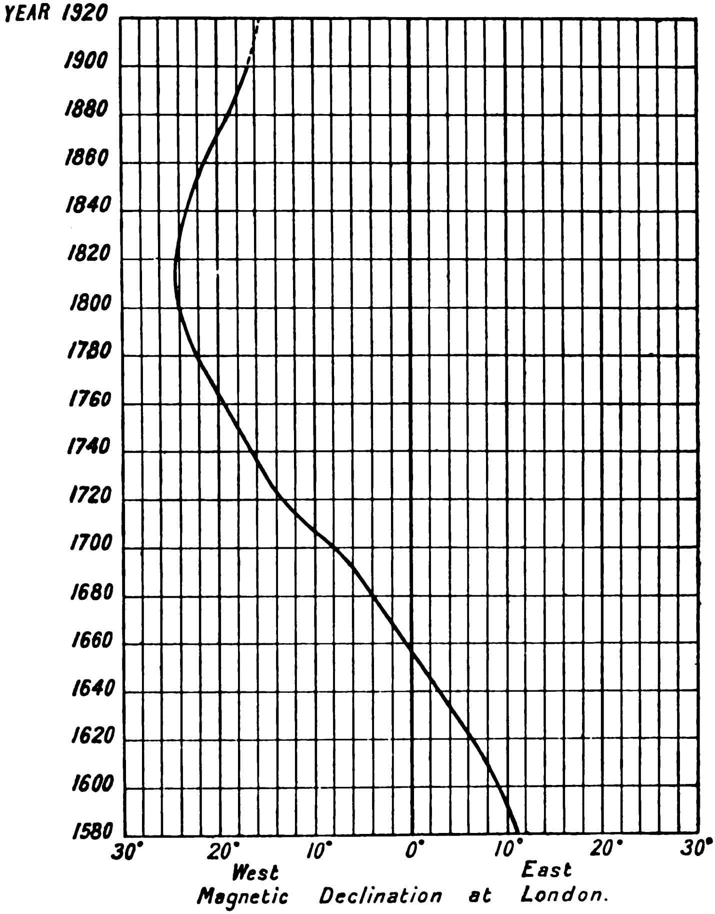

FIG. 112.—Diagram of Secular Variation at London.

causes the needle to indicate something other than the magnetic meridian at a place.

Bearing of a Line.—The bearing of a line is the angle less than 90° which it makes with a north and south direction. The line

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AB (Fig. 113) makes two supplementary angles BAN and BAS with the north and south direction NS. The bearing of the line AB is the smaller angle BAN, and would be stated as 35° east of north, or N. 35° E. Similarly, the bearing of the line AC (Fig. 114) is the angle CAS or S. 40° E., the bearing of line AD is S. 45° W., and the bearing of line AF is N. 30° W.

Magnetic Bearing is the bearing of a line with respect to the magnetic meridian at the place.

True Bearing is the bearing of a line referred to the true north and south direction, that is, to the geographical meridian at the place.

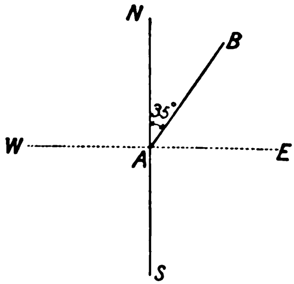

FIG. 113.—Bearing of a line.

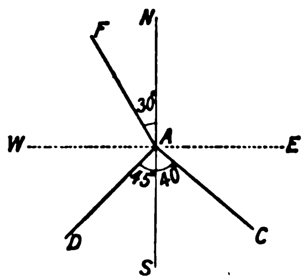

FIG. 114.—Bearings of lines.

Whole Circle Bearing or Azimuth.—The whole circle bearing of a line is the clockwise angle between the north and south direction and the line, the angle being measured round from north as the starting point. Whole circle bearings run from 0° up to 360°. Referring back to Fig. 114, the whole circle bearing of the line AC is the angle NAC, or 140°. The whole circle bearing of AD is the angle NAD measured clockwise, and is equal to 180° + 45° or = 225°, and the whole circle bearing of AF is 330°.

Forward Bearing and Back Bearing.—The bearing of a line, whether it be magnetic, true, or whole circle, differs according as the observation is made from the one end of the line or from the other. In Fig. 115 the bearing of the line AB taken from end A is seen to be N. 40° E., while taken from end B the bearing is S. 40° W. Note that the numerical value of the bearing is the same from both ends, but the designating letters are different.

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In Fig. 116 the whole circle bearing of the line CD taken from C is 105°, while taken from D the whole circle bearing is 285°, the difference being 180°. Forward bearing and back bearing are terms used to distinguish between the bearings of a line taken from the different ends. If the bearings of a series of lines, such as AB, BC, CD (Fig. 117 *a*) are taken in order at their ends, proceeding from A to D, then the bearing of each line taken at the end first arrived at will be considered a forward bearing, and the bearing taken from the opposite end will be a back bearing. The angles marked 1, 3, and 5 in Fig. 117 *a* represent the forward whole

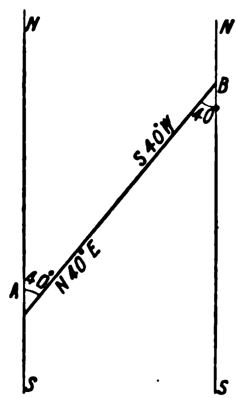

FIG. 115. — Forward and Back Bearing.

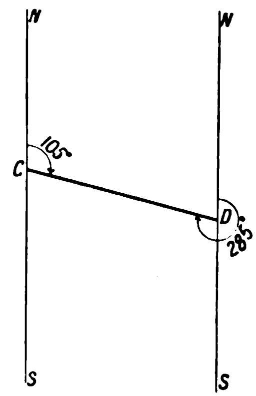

FIG. 116.—Forward and Back Bearing (whole circle).

circle bearings of the lines AB, BC, and CD, respectively, while the angles marked 2, 4, and 6 (Fig. 117 *b*) represent the back bearings of the same lines.

Surveyor's Compass.—The surveyor's compass is employed to determine the direction of a line relative to the magnetic meridian, that is, to the direction in which the magnetic compass needle points. The utility of the instrument in surveying depends on the fact that within a limited area the direction of the magnetic meridian is almost constant.

One of the forms, shown in plan in Fig. 118, consists of a circular compass box of brass or other non-magnetic metal, which is commonly 6 to 9 in. in diameter, is covered with a glass lid, and contains

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the compass needle. A circle graduated to degrees or half degrees is engraved on a raised ring round the inner periphery of the box, the graduations being at the level of the top of the needle. The cardinal points E. and W. are marked on the circle in the reverse of their normal order, that is, in looking from S. towards N., W. is to the right hand and E. to the left hand. There are two zeros, one at the N. point and one at the S. point, and each quadrant is graduated from 0° up to 90°. A pivot with hardened steel point projects from the centre of the bottom of the case and supports the compass needle. The needle can be lifted off its bearings

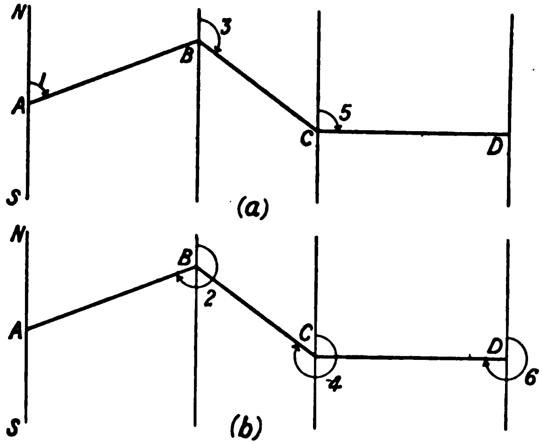

FIG. 117.—Forward and Back Bearings of Lines.

when not in use by means of a lever which passes through the side of the case. Hinged sight vanes, each having a vertical slot and a vertical hair sight, are attached to projections on the side of the case. The line of sight of the vanes is in the same plane as the north and south points of the card. In the better instruments one or two spirit levels are fixed to the box for levelling purposes. A vertical socket attached underneath to the centre of the case slips over a spindle fixed to the ball-and-socket joint at the apex of the tripod. The instrument can be rotated about the spindle and clamped in any position by a screw. The levelling of the instrument is done by turning it about the ball, which can also be clamped.

In the instruments which are suitable for longer sights and finer

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work the sight vanes will be replaced by a telescope, plate screws will be provided for exact levelling, as in the Dumpy level, and a vernier may also be provided for reading the angles.

An inspection of the figure will show why the points are marked in the reverse order. It has to be remembered that the needle points in a constant direction and that the *graduated circle* rotates with the line of sight. When the line of sight, therefore, as shown in the figure, is directed towards the N.W., the needle indicates

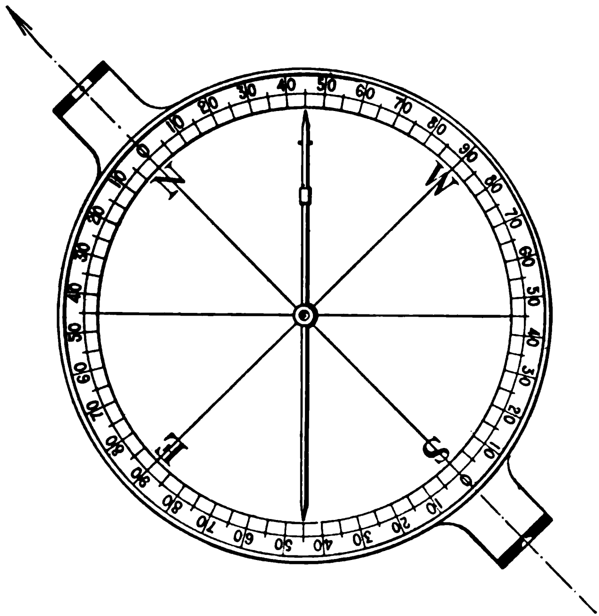

FIG. 118.—Surveyor's Compass.

N.W. on the circle as it should do. If the points were marked in their normal order, the needle would indicate N.E., which would manifestly tend to confusion.

Prismatic Compass.—The prismatic compass is the most useful form of pocket or hand compass. In it the compass card is attached to the needle, and the north point of the *card* consequently always points to the magnetic north. The graduations, as shown in Fig. 119, run from zero at the S. point to 360° in a *clockwise* direction. There is a hinged vane at one side with hair sight, and at the opposite side of the case there is a vertical sight slot combined with a triangular glass prism which magnifies the graduations and enables the card to be read at the instant the sights are in correct line. The angle read is the whole angle of the line of sight, measured round from magnetic north in a clockwise direction. The reason for commencing the graduation of the card at the S. point in order to give the required angle will be evident from an inspection of Figs. 119 and 120. The line of sight is shown pointing N.W., the

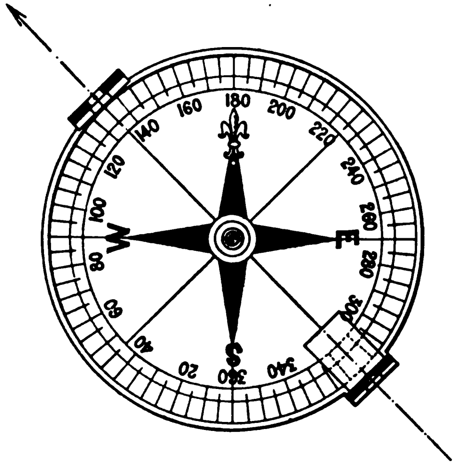

FIG. 119.—Graduation of Prismatic Compass.

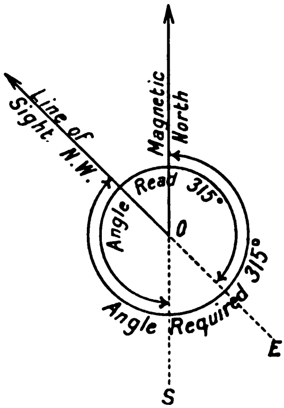

FIG. 120. — Explaining Graduation of Prismatic Compass.

whole angle of this direction measured round from magnetic north being 315°, and this is the card reading.

The prismatic compass is useful for rough and rapid work.

Compass on Theodolite.—This is shown in Fig. 121. The E. and W. points are marked in the reverse order, as in the surveyor's compass, and the graduations generally run from 0° to 360° in an *anti-clockwise* direction, starting from the N. point. The telescope is fixed in line with the N. and S. points.

Methods of Graduating the Compass Card.—In addition to the methods of graduation above described other methods are sometimes used. The surveyor's compass is sometimes graduated from 0° to 360° in an *anti-clockwise* direction, starting from the S. point,

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and sometimes it has both the complete circle and the quadrant graduation. Some of the older forms of compass have the points and graduations marked in the *clockwise* direction. Pocket compasses (not the prismatic) are usually graduated from 0° to 360° in a clockwise direction, starting from the N. point, the graduated card being attached to and turning with the needle. In using

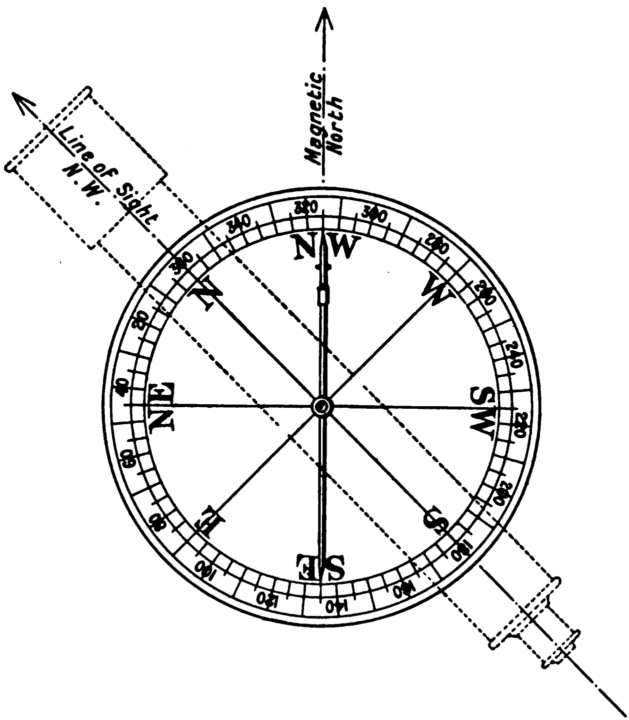

Fig. 121.—Graduations of Compass on Theodolite.

any compass the method of graduation should be very carefully studied so that there may be no confusion when the angles read come to be made use of.

Taking a Magnetic Bearing with the Compass.—To take the bearing of a line between two points, set up the compass over one of the points, level it, and clamp the ball. Let down the needle on to the pivot and turn the compass till the sights are brought into exact line with the distant point. The eye should, as far as

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possible, be always applied to the same vane, preferably the one marked S., and the north end of the needle should be used as the index. Should the eye then on occasion be applied to the north vane the correct bearing will be obtained if the reading is taken at the south end of the needle. If the needle swings violently, its vibrations may be damped by raising the lifting lever so as just to touch it, and by tapping the case the effect of friction at the pivot may be diminished. In reading the graduation hold the eye vertically over the end of the needle, as otherwise an error of 1° or 2° may readily occur in the reading. Bearings may be read to half or quarter degrees, according to the size of the circle.

Advantages of the Compass.—The chief advantages of the compass arise from (*a*) the lightness and portability of the instrument and the rapidity with which it can be set up and a bearing taken ; (*b*) the fact that bearings are referred to magnetic north and the direction of each line is determined independently of that of any other line. The relative directions of lines within a limited area can be determined, although the lines are not connected. There is no error carried forward from one line to another.

Limitations of the Compass.—(*a*) With the ordinary compass bearings are not reliable to closer than about $\frac{1}{4}°$; $\frac{1}{4}°$ of error means a deviation of about 1 ft. in a length of 230 ft., so that the compass is only suitable for very rough work.

(*b*) The needle is liable under certain conditions to give wrong indications. The bearing of a line cannot be reliably obtained in towns owing to the proximity of iron or steel and electric currents. Similarly, the needle is unreliable near railway lines, and may be affected by any small pieces of iron, such as knife blades, metal buttons, &c., carried by the observer, or by local causes within the earth.

Use of Compass Surveying.—Compass surveying is only suitable on an extensive scale for rough work, where a considerable degree of accuracy is not expected or required. It, however, furnishes a very useful method of carrying out rapid preliminary work, such as for the approximate location of roads, railways, &c., in new countries, and is extensively used in mining work.

It may also often be used to advantage in locating the details

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of a survey whose main lines have been fixed by some more accurate method. Where numerous short bearings are required, the work will be much more rapidly overtaken with the compass than with the theodolite, and, provided care is taken to limit the extent of work surveyed independently by the compass, quite good results may be obtained.

Compass Surveying with Needle only (Free Needle).—A system of survey lines is laid out in convenient proximity to the objects to be located. The arrangement of lines may take any shape, there being no limitation to a particular form, such as is necessary in chain surveying. A narrow strip of ground, such as a stretch of roadway, would be surveyed from a series of connected lines laid out in lengths to suit the windings of the road. In this case the accuracy of the work is mainly dependent on the accuracy with which the bearings and lengths are measured. There is nothing in the arrangement of the work to enable a mistake in measurement or bearing to be detected during plotting.

A single enclosure would be surveyed from a set of lines forming a closed polygon. The lines may run around, within or along the boundaries as best suits the case.

The lines to survey a considerable area should, where possible, be arranged in a series of closed polygons.

Objects are referred to the survey lines by any of the methods described under chain surveying, and the compass itself may be used to furnish an additional method. By the latter method, which is very convenient for objects at some distance from the survey line, the compass bearing is taken to the object from a point on the survey line and the distance is also measured. Objects located in this way by bearing and distance are plotted by protractor. As the general accuracy attainable in compass surveying is not great, refinement in locating details is unwarranted.

The bearings of all the lines of a polygon may be obtained by setting up the instrument at each alternate angle. Thus, in the polygon ABCDEF (Fig. 122) all the bearings could be obtained by planting the compass at the points A, C, and E, the bearings of two sides being read from each point, and if the direction of the magnetic meridian could be relied on as being constant throughout, it would not be necessary to set up the compass at the other three points.

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There would, however, be nothing to show whether the indications of the needle were affected by local attraction at any point. By setting up the compass at all the angles the bearing of each line will be determined from both ends. If the bearing of a line taken at one end is not the exact converse of the bearing taken at the other end, the difference is probably due to local attraction.

For methods of plotting the survey lines of a compass survey see Chapters XII. and XIII.

Correction for Local Attraction.—Local attraction will affect equally the bearings of any two lines taken from the same point. Hence the correct angle between two lines can be obtained at their point of junction, even although their magnetic bearings are individually incorrect. This points to a method of correcting the bearings in a system of lines where it is evident that local attraction occurs at only a few points. The table on p. 114 shows a set of bearings referring to Fig. 122.

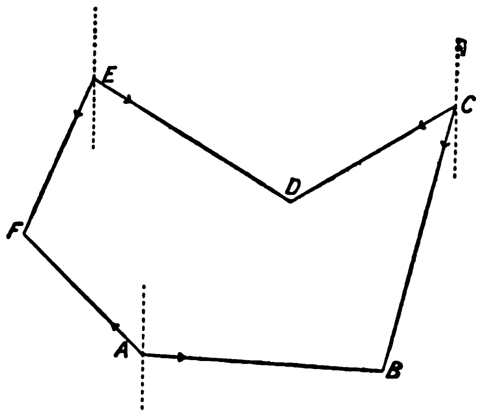

FIG. 122.—Compass Bearings of a Polygon.

An inspection of the forward and reverse bearings of the various lines shows that there is evidently no local attraction at the points A, B, C, and F. The bearings of the lines C D and FE taken from the points C and F respectively may, therefore, be accepted as correct. The bearing of DC taken from D is 4° different from the converse bearing taken from C. The bearing of DC is, therefore, corrected to correspond with the bearing of CD. The bearing of DE must then be altered by the same amount as DC, and in such a way as to leave the whole angle CDE unaltered. The bearings of the other lines affected are dealt with successively, the corrections required being as shown in the table.

TABLE SHOWING CORRECTION OF MAGNETIC BEARINGS FOR LOCAL ATTRACTION.

Line.	Observed Bearing.	Correction.	Corrected Bearings.
AF	N. 47° W.	—	N. 47° W.
AB	S. 85° E.	—	S. 85° E.
BA	N. 85° W.	—	N. 85° W.
BC	N. 15½° E.	—	N. 15½° E.
CB	S. 15½° W.	—	S. 15½° W.
CD	S. 60° W.	—	S. 60° W.
DC	N. 64° E.	— 4°	N. 60° E.
DE	N. 55½° W.	+ 4°	N. 59½° W.
ED	S. 57° E.	+ 2½°	S. 59½° E.
EF	S. 27½° W.	— 2½°	S. 25° W.
FE	N. 25° E.	—	N. 25° E.
FA	S. 47° E.	—	S. 47° E.

Booking the Survey.—It is most convenient to have the bearings recorded on a well-conditioned sketch of the survey lines. The stations are numbered or lettered and the details of each line are booked separately, as described under chain surveying. It is of much assistance in plotting to have the lengths of the survey line figured on the general sketch.

As magnetic north is not a fixed but a varying direction, it is important that the magnetic declination or variation of magnetic north from true north should be accurately determined and recorded on the plan, together with the date of the survey. In newly-settled countries the preparation of survey plans and description of boundaries with reference to the magnetic north direction only has given rise to frequent disputes in later years owing to dubiety arising as to the true magnetic declination at the time and place of the survey.

Surveyor's or Box Sextant.—This is a very useful hand instrument for measuring angles up to about 120°. It possesses the serious limitation for surveying purposes that it can only be used to give accurate results on ground which is level or nearly so. Fig. 123 illustrates the principle of the instrument, and Figs. 124 and 125 show a top view and horizontal section respectively. Two mirrors

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are employed, as in the optical square, to bring two separate objects into view simultaneously. A fixed mirror, A, and a movable mirror, B, are attached to the underside of the lid of the circular box frame of the instrument. The movable mirror, B, is connected to a spindle, which passes through the top of the case and carries the index arm which gives the angle on the graduated circle EF.

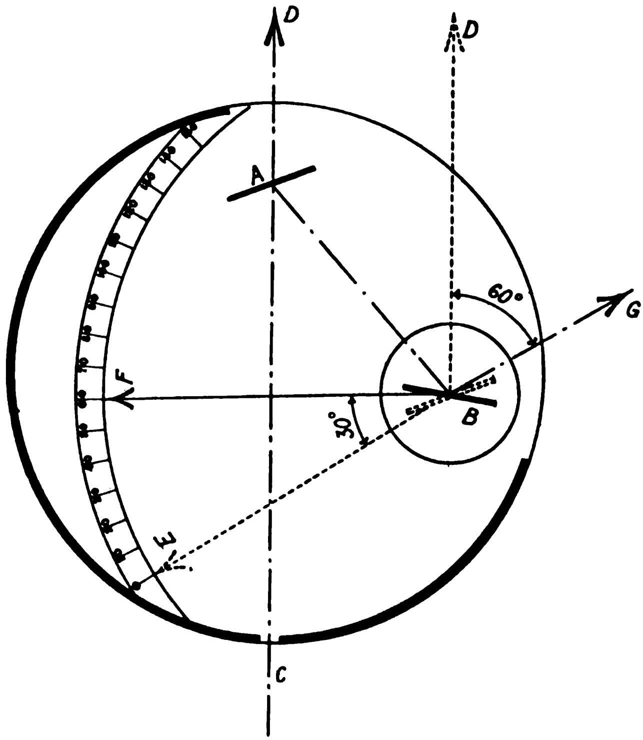

FIG. 123.—Principle of Box Sextant.

An eyehole at C in the side of the case is opposite a larger hole in the opposite side, and through these one of the objects, D, is seen directly over or under the mirror A. To bring the other object, say, G, into view in mirror A, mirror B must be rotated. Lines GBAC represent the path of a ray of light from object G when the image of the latter is seen in A. When mirror B is parallel to mirror A, as shown in dotted lines, the index arm points to the zero of the graduated circle, EF, and the reflected ray BD, from an object D,

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is parallel to the direct ray CD from the same object. The reflected image of D, therefore, appears in coincidence with the object seen directly. When the image of G appears in coincidence with D the angle included between the lines from the sextant to the two objects is GBD. As shown for the optical square, the angle GBD is double the angle EBF, through which the mirror B and the index arm turn. The graduated arc EF is, therefore, divided so that each $\frac{1}{2}°$ is figured as 1°, or there are 180° to the quadrant instead of 90°. The whole instrument is contained in a case about 3 ins.

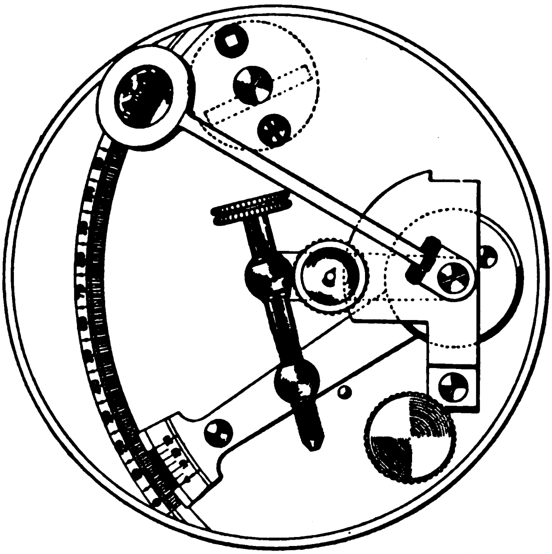

Fig. 124.—Top View of Box Sextant.

diameter. The index arm is furnished with a clamp and tangent screw for fine adjustment and has a vernier reading to single minutes. A small lens attached to a hinged arm enables the vernier to be read. Two darkened glasses can be interposed singly or together between the mirrors when readings are being taken to the sun. Provision is made for adjusting the fixed mirror A. It can be rotated a small amount in the horizontal direction by means of a square-headed screw in the side of the case, and a similar screw in the top of the case serves to adjust it vertically. A key to fit the heads of these screws is attached to the top of the case. A

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metal cover screwed on over the case serves to protect it from injury when not in use.

Testing the Sextant.—(*a*) Testing for position of the image. To test whether the mirror A shows the image in a position favourable for accurate observation, set the index arm to zero, look directly

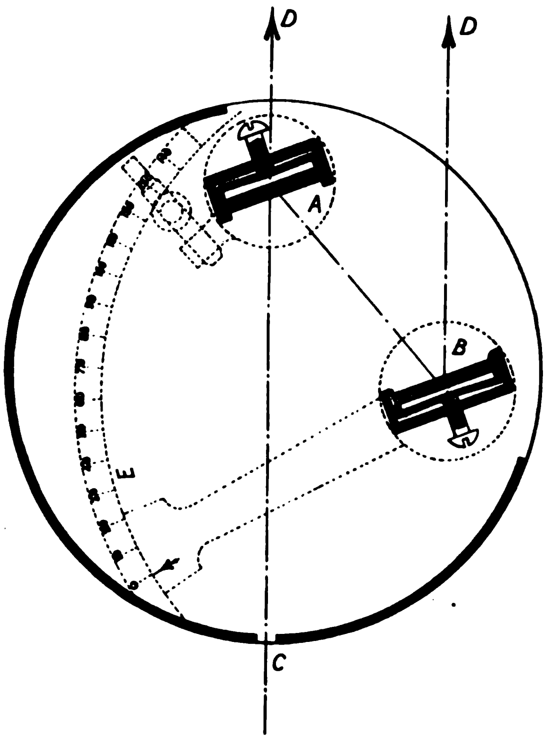

FIG. 125.—Horizontal Section of Box Sextant.

at an object and observe the position of its image in mirror A. If the image appears high above the object the instrument cannot be accurately used. Also, if when the image is visible the object itself is hidden by the mirror, the instrument is unusable. In either case, the mirror A requires to be adjusted vertically. To do this turn the square-headed screw in the top of the case until the image and object are seen simultaneously with the smallest possible vertical interval between them.

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(*b*) Testing for zero error. Set the index accurately to the zero graduation, hold the instrument level and sight directly to a distant definite object, preferably a vertical line, such as a ranging pole or the corner of a house. If the instrument is correct, the portion of the object seen in the mirror will appear in continuous vertical line with the portion seen directly. If the image appears slightly to one side of the object the instrument is in error at the zero point. To correct this apply the key to the screw in the side of the case and rotate mirror A until the image appears in exact vertical coincidence with the object. The sextant then indicates correctly at the zero point.

(*c*) Testing for error at 90°. To test whether the instrument is correct at 90° set the index to 90 and use it as an optical square to set out two right angles from the same intermediate point on a straight base line, as explained on p. 19. If the two lines as set out coincide, the sextant is correct at 90°. If there is found to be considerable error at 90° when the zero is correct, the instrument is faulty and cannot be made right by the surveyor.

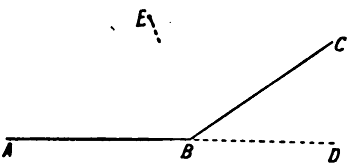

FIG. 126.—Measuring large Angle with Sextant.

The sextant may be tested throughout its range by comparing the readings of various angles with those given by a reliable theodolite. A faulty sextant may be used to give accurate results if its error is found at intervals throughout its range and the readings are corrected accordingly.

Advantages of the Sextant.—The advantages of the sextant lie principally in its portability and accuracy, and the rapidity with which angles can be read. On *level* ground a 3-in. diameter sextant is much more accurate than any form of compass.

Limitations of the Sextant.—(*a*) Smallness of the angle which can be read. The largest angle which can conveniently be read with the sextant is 120°. If the size of an angle greater than 120°, such as ABC (Fig. 126), is required, the line AB might be produced and marked at D, and the angle CBD read with the sextant. The required angle ABC would then be equal to 180° − CBD. Another method is to fix on some object E dividing the angle into two

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portions, and read first the angle ABE and then the angle EBC. The sum of these angles gives the required angle ABC.

(*b*) Accurate use is limited to ground which is nearly level. The sextant measures the actual angle between two lines in the plane containing the lines, while the angle required for surveying purposes is the horizontal projection of the actual angle. On sloping ground an angle measured with the sextant may be greater or less than the true horizontal angle.

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CHAPTER X

THE THEODOLITE

This chapter deals with the principles of construction of the theodolite, and with its use in the measuring of horizontal and vertical angles, ranging straight lines between survey stations, prolonging straight lines, and ranging survey lines between stations which are invisible from each other.

Theodolite.—The theolodite is used by the surveyor for the purpose of accurately measuring horizontal and vertical angles, and also for setting out angles on the ground, ranging straight lines, setting out curves, and for setting out the lines of intended works. It is the most important instrument used by the surveyor, and one with the principles of whose construction and use he must make himself thoroughly familiar if he is to become expert at his work.

Before describing the theodolite in detail we shall indicate shortly its essential elements so as to give a general idea of the instrument as a whole (see Fig. 127). We may consider it as consisting of two main portions: the support or stand, usually in the form of a tripod, required to bring the instrument to a convenient height for the observer's eye; and the upper or working portion. The upper portion is connected to the stand by three or four levelling screws, which permit of its being levelled up so as to make the axes of the rotating parts of the instrument truly vertical and horizontal. Immediately above the levelling screws there is a horizontal circle graduated to degrees and sub-divisions. This circle can rotate and can be fixed in any position by a clamp. Directly above the graduated circle and concentric with it there is a vernier or index circle which carries the standards supporting the telescope. The index circle, standards and telescope can rotate together horizontally on top of the graduated circle. The support of the telescope is by means of a horizontal axis resting on top of the standards so that the telescope can rotate independently in a vertical plane.

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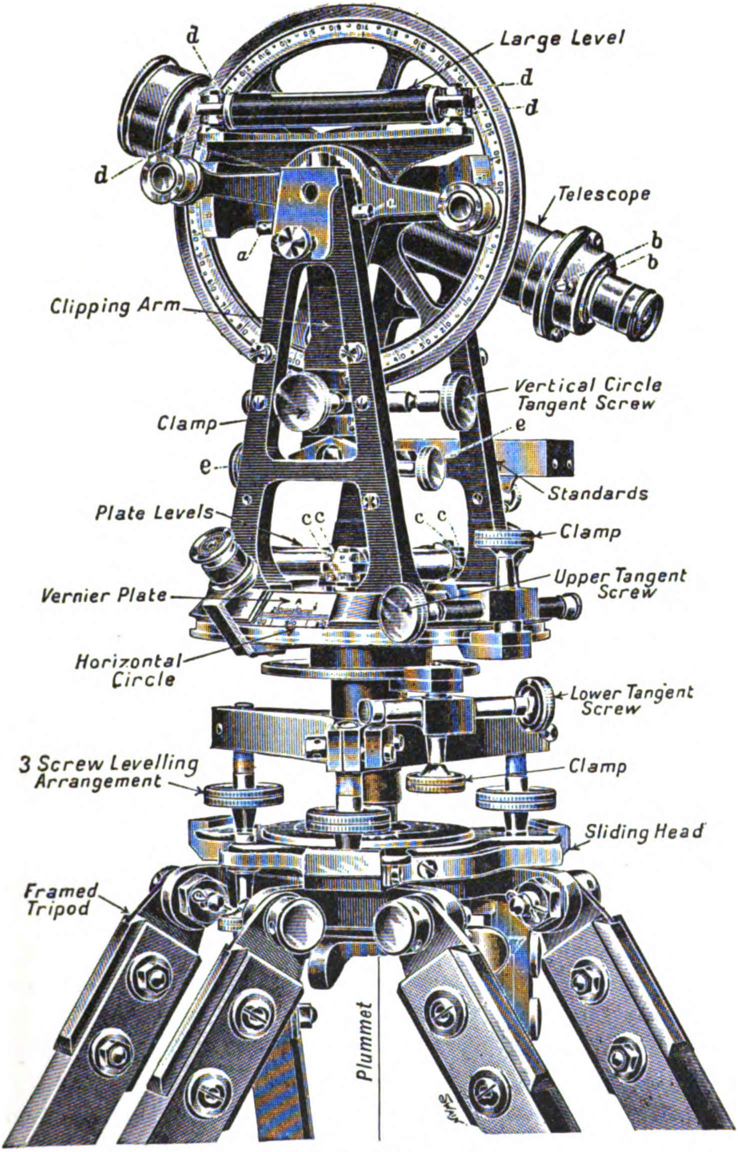

FIG. 127.—Theodolite.

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For the reading of vertical angles there is a vertical graduated circle which is attached to the telescope and rotates with it. The vertical circle is read by means of a fixed index arm.

A clamp and slow motion tangent screw are provided for controlling the motion of the whole upper working portion of the instrument, including the horizontal circle, and similar arrangements are provided for controlling the motion of the vernier circle and the parts above it relative to the horizontal graduated circle, and for controlling the motion of the telescope and vertical circle in the vertical plane.

A pair of spirit levels at right angles to each other are provided on the vernier plate, or sometimes one on the vernier plate and one on a standard to show when the instrument is properly levelled. For use in reading vertical angles it is necessary to have a large spirit bubble attached either to the telescope or over the index arm of the vertical circle.

The size of a theodolite is the diameter of its horizontal graduated circle in inches. Modern theodolites are mostly confined within the limits of the 4-in. and 10-in. size; the most common size for ordinary use being the 5-in. reading to single minutes.

In the form of theodolite known as the transit, which is the most common form, the standards are high enough to permit of the telescope making a complete revolution in the vertical plane, or of being "transited," as it is called. For the sake of compactness the standards are sometimes made so low that the telescope cannot turn right over or cannot be "transited," a restriction which greatly curtails the usefulness of the instrument for purposes of ranging out lines and setting out works. In the non-transiting form of theodolite (plain theodolite) the complete vertical circle is replaced by either a single graduated arc under the telescope with a single vernier, or by two graduated arcs of a circle diametrically opposite each other with a vernier to each.

Support or Stand.—The most common form of support or stand for the theodolite consists of a tripod formed of three tapering solid wooden legs, pointed and shod with iron at the lower ends and hinged at the top to a brass or gun-metal casting, which is screwed to receive the upper portion of the instrument. The legs are of rounded triangular section, so that they take up a circular form

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when closed. They are bound together by brass rings or leather straps when not in use, and a cap is provided to screw on to the head of the tripod and protect the threads from injury.

In situations where the tripod cannot be set up a support of the form shown in Fig. 128 is sometimes useful. It may be set on the level top of a wall or a level board may be readily fixed up to carry it in places where much trouble would be required to construct a stand for the tripod.

The theodolite illustrated in Fig. 127 is supported by the framed type of stand, which, when properly and substantially constructed, is preferable to the solid wooden type in respect of stiffness. In this type each leg of the tripod consists of two members, which incline downwards towards each other, and are joined together at the bottom and shod to form a single point of support. The two members forming a leg are braced together at one or more intermediate points. The framed type of stand is almost universally used for large theodolites.

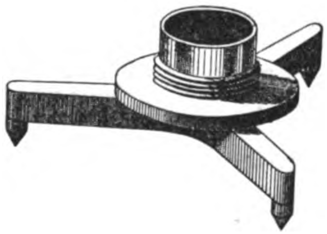

FIG. 128.—Triangular Stand.

Parallel Plates.—The lower parallel plate either screws on to or forms part of the head of the tripod, and is a fixed portion of the instrument. In the four-screw form of construction the connection between the upper and lower plates is through a ball-and-socket joint and the thumb-screws. These are screwed into the upper plate and simply rest on the lower plate, the foot of one of them fitting into a cup to prevent rotation of the upper plate.

The upper parallel plate contains the socket which receives the spindle supporting the whole rotating portions of the instrument. The thumb-screws provide the means of levelling the upper portion of the instrument and rendering its vertical axis truly vertical. In modern instruments the parallel plates are generally castings with three or four arms for the fixing of the levelling screws, and have little resemblance to plates.

Three-Screw Levelling Arrangement.—In the levelling arrangement illustrated in Fig. 127 there are three thumb-screws at 120°

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apart. In this case the screws are the only connection between the upper and lower plates, and themselves serve the function of a ball-and-socket joint. At the foot of each screw there is an enlarged ball or cone which engages in and is held by a groove or recess in the lower plate in such a manner as to permit of the screw tilting to the required extent. The upper ends of the screws are threaded and engage with the arms of the upper plate. The three-screw levelling arrangement is preferable to that with four screws. In the four-screw type, racking and straining of the screws will readily occur unless care is taken to turn each of a diagonally opposite pair at uniform rates in opposite directions. A single screw cannot be separately screwed down without causing strain. In the three-screw type any single screw can be turned (within reasonable limits, of course) without straining the instrument, and the levelling can be completed by turning the screws one at a time with one hand if necessary.

Graduated Horizontal Circle.—The graduated horizontal circle is attached to the spindle which rotates within the socket of the upper parallel plate. The graduations are engraved on a bevelled ring of silver laid round the rim of the circle. In theodolites for ordinary purposes, such as that illustrated in Fig. 127, the circle is generally 5 ins. in diameter. The graduations are $\frac{1}{2}°$ apart and run from 0° to 360° right round the circle in a clockwise direction. Attached to the upper plate there is a clamp with locking key which engages with a portion of the spindle connected to the graduated circle. The attachment of the clamp to the upper plate is through a tangent screw, so that when clamped the circle can still be turned a small amount under the exact control of the observer.

Vernier Plate.—The vernier or index plate is concentric with, and rotates on top of the graduated circle. It usually has two indexes at the opposite ends of a diameter with verniers, enabling the circle to be read to single minutes, and is frequently arranged to cover and protect the graduated circle except for a small portion opposite each index. The vernier plate can be clamped to the graduated circle, and the connection is made through a slow-motion tangent screw which enables the index to be set accurately to any desired angle. Magnifying glasses are provided for reading the verniers.

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The vernier plate carries on its upper surface the two standards of A-frame or other form which support the horizontal axis of the telescope, and two spirit levels, one placed parallel to the horizontal axis of the telescope and the other at right angles thereto. The latter spirit level is sometimes carried on one of the standards instead of on the upper plate. The spirit levels are attached to the plate or standard by capstan screws, *cc*, which enable them to be adjusted so as to indicate correctly.

A magnetic compass with a small circle graduated as described on p. 109 is usually also carried on the upper plate between the standards. Instead of the compass on the upper plate a trough compass with a long and sensitive needle is sometimes fixed to the underside of the graduated circle of the theodolite. The trough is a narrow box with a very short portion of graduated arc at each end. The line of sight of the telescope is parallel to the line joining the zeros of these arcs, so that the direction of the magnetic meridian is obtained when the needle indicates zero.

Standards.—The horizontal axis of the telescope rests in V-shaped bearings on the top of the standards and is held down by hinged clips fastened by thumb-screws. One of the bearings can be adjusted vertically by means of the screws *a*, *a*.

Arrangement of Telescope, Vertical Circle, &c.—The telescope is fixed at right angles to its horizontal axis, and the vertical circle is rigidly attached to the telescope and rotates with it. The verniers are at the end of a horizontal arm which is formed in one piece with a vertical clipping arm, the latter being attached to a projection on one of the standards by means of opposing screws. The horizontal axis of the telescope passes through the junction of the clipping arm with the vernier arms. A clamp and tangent screw attached to the vertical arm serve to fix the vertical circle when required and give it a slow rotation for accurate setting of the telescope. The complete vertical circle attached to the telescope of a transit theodolite has two zeros, which can be made to coincide with the indexes of the verniers when the line of sight is horizontal, each quadrant being graduated from 0° up to 90°. Sometimes there are four zeros at the extremities of two diameters at right angles to each other, and each quadrant is then divided up from 0° to 90° in a clockwise direction. The vertical circle is also

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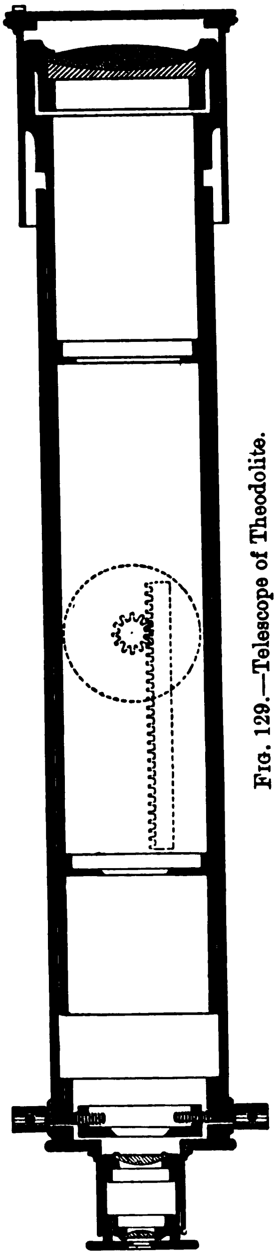

Fig. 129.—Telescope of Theodolite.

sometimes graduated from 0° to 360° in a clockwise direction, giving 0° at one vernier and 180° at the other when the telescope is level. With this method of graduation angles of elevation can be distinguished from angles of depression without remark if the telescope is not transited.

A sensitive spirit level is attached to the upper or under side of the telescope, or, as shown in Fig. 127, to the vernier arm. The line of sight is arranged parallel to the axis of the bubble tube (see Chap. XVI.), so that it will be horizontal when the bubble is at the centre of its run. The attachment of the spirit level to the telescope is by means of opposing capstan screws which permit of its accurate adjustment.

Telescope.—The essential parts of the telescope are the tubes, the object glass, the magnifying eyepiece, and the diaphragm with hair sights. The arrangement of the telescope is shown in Fig. 129. The object glass is fixed in the end of a tube which slides within the main telescope tube and has a range of motion of 1 or 2 ins. The eyepiece also slides within a contracted portion of the main tube, with a very much smaller range of motion than the object glass. Frequently, however, the object glass end is fixed and the motion for focussing takes place at the eyepiece end. The diaphragm is fixed in the main tube near the eyepiece end. The object glass or eye end, as the case may be, is moved in and out by turning a thumb screw on

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the side of the telescope, which works a rack and pinion inside the tube. The object glass is a combination of two lenses placed in contact. The front lens is double convex and made of crown glass. The back lens is of flint glass and plano-concave, that is, one side is plane and the other is hollowed to fit against the front lens. The combination has very nearly the same effect as a single double-convex lens, but gets rid of certain optical imperfections which are unavoidable with the single lens. Rays of light coming from a *point* in front of the object glass and passing through it converge to a *point* in the vicinity of the diaphragm. Rays of light from an *object* in front of the object glass converge on passing through it and form a small inverted image of the object near the diaphragm. The focussing of the object glass consists in moving it or the eyepiece end in or out until this image coincides with the cross hairs.

The eyepiece generally consists of two small plano-convex lenses, placed as shown in the figure, the combination forming a microscope with which the cross hairs and coincident image of the object are viewed. With such an eyepiece the object appears inverted, that is, it appears upside down and the right-hand side appears to the left hand. To focus the eyepiece on the cross hairs, sight the telescope towards the sky and move the eyepiece in or out until the cross hairs appear perfectly distinct. It should then be in correct focus, and should not require to be again altered for any sight taken by the same observer. When the eyepiece has been correctly focussed on the cross hairs, sights may be taken with the telescope. To bring an object into view, start with the object glass in its furthest in position and turn the thumb screw to move it slowly outwards. A point will be reached at which the image appears brightest and most distinct, and the object glass is then correctly focussed. The cross hairs and image should now both appear clear and definite, and if the eye is moved from side to side the cross hairs should not appear to move relative to the object. If there is any apparent motion the focussing is imperfect, but very slight motion of the object glass should now suffice to give the desired result. Apparent motion, caused by the image and cross hairs not being in the same plane or by the cross hairs not being in the focus of the eyepiece, is known as parallax. Accurate work cannot be done unless the parallax is got rid of, as otherwise the angle read would depend on the position of the eye.

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The diaphragm most commonly consists of a brass ring within the telescope tube, across the aperture of which three spiders' webs or very fine platinum wires are stretched. These are known as the cross hairs. One is horizontal and the other two are equally inclined to the vertical, one on each side of it, as shown in Fig. 130. They cross each other at the centre of the aperture. Sometimes fine lines scratched on glass or platinum-iridium points are used as a substitute for the hairs. The diaphragm ring is fixed inside the tube by two or four screws, whose capstan heads appear outside the tube. These screws pass loosely through enlarged holes in the tube and are screwed into the diaphragm ring, affording the means of adjusting the latter slightly in the vertical or horizontal directions.

The "line of sight" of the telescope is the line joining the intersection of the cross hairs to the optical centre of the object glass. The motion of the centre of the object glass in focussing should be along that line, which should pass as nearly as possible through the optical centre of the eyepiece. The term "line of collimation" is often used with the same meaning as "line of sight."

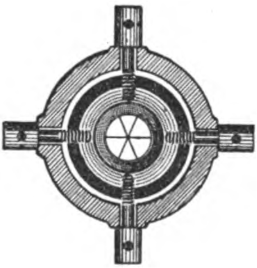

FIG. 130. — Diaphragm and Cross Hairs.

Vernier.—The vernier, so called from the name of its inventor, is a device which enables a scale to be read accurately to a small fraction of its smallest division. Its practical utility is largely due to the extreme accuracy with which scales can be machine divided, and depends also on the fact that the eye can judge very accurately when a line on one scale is exactly opposite a line on another when their graduated edges are in coincidence. The vernier consists of a short graduated scale whose edge slides against the edge of the scale to be read. The length of the vernier scale depends on the fineness with which it is desired to read the main or fixed scale. In order that the main scale should be read to the tenth part of its smallest division, the vernier scale is made exactly equal to nine of the main scale spaces and is divided into ten equal parts. To read to the thirtieth part of the main scale divisions the vernier scale would have thirty equal divisions in a length equal to twenty-nine main scale spaces.

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Fig. 131 illustrates a vernier to read to the tenth part of the scale divisions. As shown in (1) the vernier has a length equal to nine scale spaces, is divided into ten equal parts, has an index at one end, and has its graduations numbered, the zero being at the index. The vernier is indicating 5·00 on the scale. Each main scale space has a length of 0·10 unit. Each division of the vernier scale has a length equal to $\frac{0{\cdot}90}{10}$, or 0·09 unit. The difference in

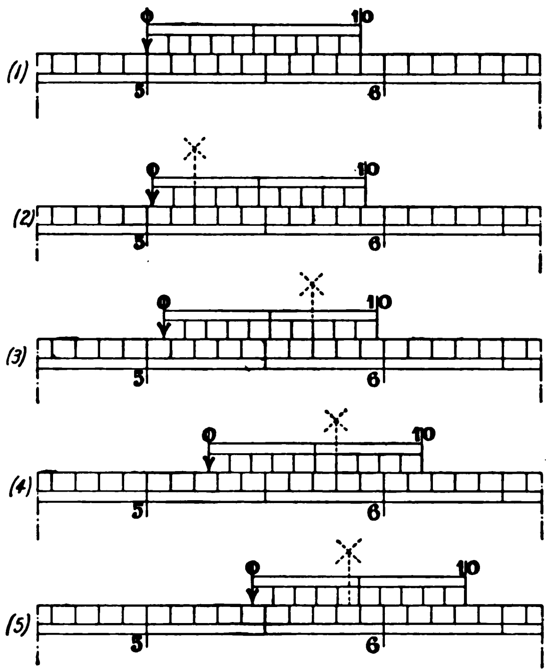

FIG. 131.—Vernier Reading to Tenths.

length between a main scale division and a vernier scale division is, therefore, = 0·10 — 0·09, or = 0·01 unit. When the vernier index then coincides with the line marked 5 on the scale, the next line on the vernier will be 0·01 short of the next line on the scale, the second vernier line will be 0·02 short of the second scale line past the number 5, and so on. If, therefore, the index is moved forward 0·01 unit, or one-tenth of a scale division, the vernier line, No. 1, will come opposite a scale division, and the scale reading is then 5·01. If, as shown in Fig. 131 (2), the index has been moved forward two-tenths of a division from the line marked 5, the vernier line No. 2 will coincide with a scale division, and the reading will be 5·02. Fig. 131 (3) shows vernier line No. 7 in coincidence with a scale line, and the reading is 5·07. The index in Fig. 131 (4) records 5·2 *plus* a fraction on the main scale. Vernier mark No. 6 is opposite a scale mark. The scale reading is, therefore, 5·26.

Any other vernier is read in a precisely similar manner.

It may happen that no one line on the vernier is exactly or nearly opposite a scale line, but that two adjacent vernier lines appear equally close to two lines on the scale. This is illustrated in Fig. 131 (5), where the two vernier lines, Nos. 4 and 5, are symmetrical with respect to two of the scale lines. In such a case the average is taken, the reading indicated being 5·445.

Theodolite Verniers.—In the graduated circle of the ordinary 5-in. theodolite the lines are engraved at $\frac{1}{2}°$ intervals, so that each space represents thirty minutes. The vernier reads to single minutes, or one-thirtieth of the smallest scale division, and therefore its length is made equal to twenty-nine circle spaces, and this length is divided into thirty equal parts.

A theodolite vernier reading to minutes is illustrated in Fig. 132. In Fig. 132 (1) the length of the vernier scale is seen to be equal to $14\frac{1}{2}°$ or twenty-nine spaces of the circle. The reading is 20° 0′. Fig. 132 (2) shows the index pointing between 22° and $22\frac{1}{2}°$, and the vernier mark, No. 12, is opposite a scale graduation. The reading indicated is, therefore, 22° 12′. Fig. 132 (3) shows the index pointing between $25\frac{1}{2}°$ and 26°, and vernier mark No. 27 coincides with a scale mark. The reading is, therefore, 25° 30′ + 27′ or 25° 57′. A common mistake when one is intent on reading the vernier correctly is to neglect to add the $\frac{1}{2}°$ in angles such as the latter, the result being that 25° 27′ is booked instead of 25° 57′.

In theodolites with circles larger than 5 ins. diameter the graduations are usually at intervals of $\frac{1}{3}°$, and the vernier reads to one-third of a minute or twenty seconds. The vernier, therefore, reads to the one-sixtieth part of the smallest circle space and requires to have sixty divisions in a length equal to fifty-nine circle spaces. Every third mark on the vernier scale represents a whole minute

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and is drawn longer than the others. These lines representing minutes are numbered from 0 up to 20, and the intermediate short lines represent twenty seconds or forty seconds. Fig. 133 shows

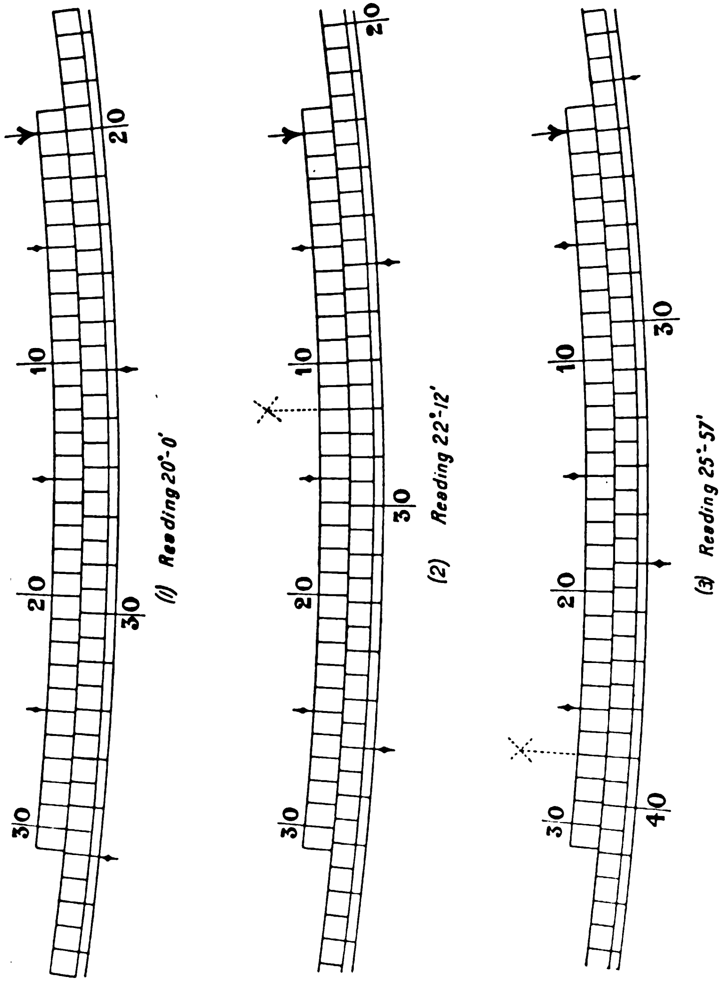

FIG. 132.—Vernier Reading to Minutes.

a vernier reading to twenty seconds, drawn much larger than actual size. In Fig. 133 (1) the reading is 30° 0′, and it is seen that the vernier scale covers 19° 40′ of the circle. In Fig. 133 (2) the reading is 30° 12′. In Fig. 133 (3) the index is one space and a fraction past

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the whole degree mark and twenty minutes must, therefore, be added to the vernier reading. The reading shown is 32° 20′ + 7′ 40″, or 32° 27′ 40″. In Fig. 133 (4) the index is two spaces and a

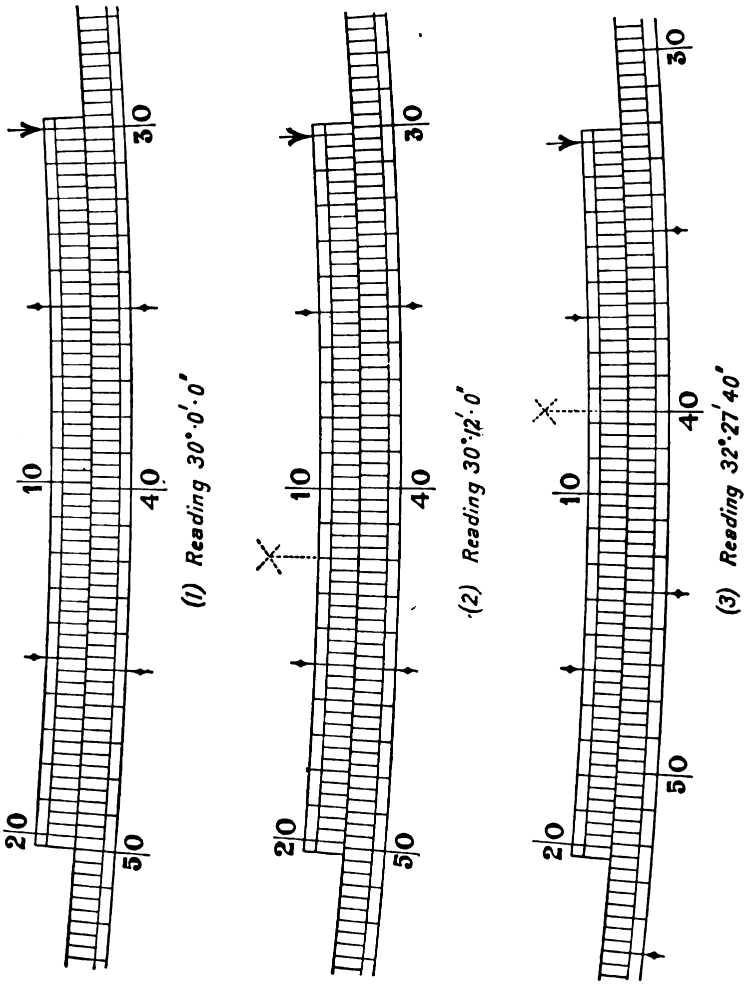

Fig. 133.—Vernier Reading to 20 Seconds.

fraction past the whole degree mark and forty minutes have to be added to the vernier reading. The reading shown is 35° 40′ + 13′ 20″ or 35° 53′ 20″. With this vernier care must be taken not to drop 20′ or 40′ from the correct reading.

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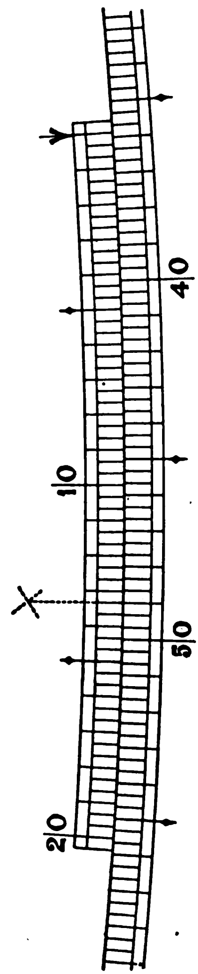
In the vernier scales shown in Figs. 132 and 133 there is an extra mark at each end beyond the scale proper. These extra marks facilitate the reading when the index is nearly opposite a scale division, and are specially useful in setting the index to zero or to a given angle. The index can be most accurately set by looking at the marks on each side of it, and noting when these are exactly symmetrical with respect to the adjacent scale marks.



(4) Reading 35°53′20″

Fig. 133.—Vernier Reading to 20 Seconds.

Before reading any vernier it is a good plan to estimate directly from the position of the index on the scale the approximate number of minutes. This can be done to within a few minutes, and forms a rough check, which tends to the avoidance of large mistakes. At the same time it enables the vernier to be read more rapidly, since the approximate position of the mark is known, and it need not be looked for elsewhere.

There are two verniers diametrically opposite each other on most theodolites. Only for work of great precision are both verniers made use of, and then in the manner explained in Chapter XIV. In ordinary work only one vernier is read, and it is important that the same vernier should be used throughout, as the verniers are not always exactly 180° apart. The two verniers are generally distinguished by the letters A and B. Always use the A vernier. Use the B vernier only when both are being read.

Use of the Theodolite.

Use of the Theodolite.—In using the theodolite the preliminary operations consist of setting up the instrument over a given point, centering it exactly over the point by means of the plumb-bob,

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and thereafter levelling it by means of the plate screws and spirit levels.

Setting up the Theodolite.—The parts of the head of the theodolite are carried in a box which is accurately fitted and padded to prevent any looseness or motion of the parts. There is usually only one definite position and state of adjustment in which the parts will go into their allotted places. It is therefore very important, before taking the portions of a theodolite out of the box for the first time, to make a careful note of their position and arrangement in the box. It will be found very useful to make a sketch of the arrangement on the inside of the lid of the box. The 5-in. theodolite, without sliding head, is usually packed into the box in two main portions. The lower portion which screws directly on to the tripod head comprises the parallel plates, the horizontal graduated circle, the vernier plate, and the standards. The upper portion consists of the telescope vertical circle, &c.

Having set up the tripod and unscrewed the protecting cap, lift the lower portion of the instrument carefully out of the box and screw it on to the tripod head. Bring all the plate screws to a bearing on the lower plate, see that they are equally screwed up, and then open the clips over the telescope bearings at top of the standards. Take now the telescope portion, bring the vertical clamping arm between the standards and on to its attachment, at the same time lowering the horizontal axis on to its bearings. Bring the opposing screws of the vertical clipping arm attachment to a hold and fasten the clips over the ends of the horizontal axis.

The theodolite is carried from place to place on the shoulder. To avoid straining the instrument from the unavoidable jolting in carrying it, the lower clamp should be loose, so that the whole head may be at liberty to rotate. The vernier plate clamp may also be loose. The telescope should be pointed vertically upwards and clamped. In passing under trees with low branches or through low passages the theodolite should be carried under the arm with the head in front.

Setting over a Point.—See that the upper and lower plates are nearly parallel, and, if necessary, turn the thumb-screws to effect this. See also that the connections of the legs to the tripod head are sufficiently stiff. These connections in some makes of instru-

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ment are apt to become loose when the theodolite is in constant use, and accurate work is then impossible. A screw-driver or key should be kept handy for tightening the screws. Sometimes the cap of the tripod is formed as a key to fit the nuts.

The plumb-bob is suspended from a hook in the central axis of the instrument and a sliding knot or other arrangement, such as illustrated in Fig. 134, is employed in adjusting its height. The knots are shown open for the sake of clearness in illustration, but in practice they are pulled tight enough to prevent the plumb-bob from falling under its own weight. The arrangement shown in (*b*), Fig. 134, is perhaps the simplest. Both the loop and the free end of the string are pulled down vertically so that the portion carrying the plumb-bob is gripped with sufficient tightness to prevent it from slipping. In the sliding button arrangement in (*c*) an actual button may be used, or a flat piece of metal, wood, or leather with holes for the attachment and passage of the string will equally serve the purpose.

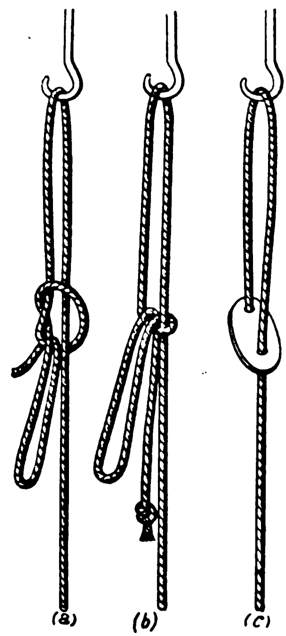

FIG. 134.—Suspension of Plumb-bob.

To bring the instrument nearly into position over a station it will be found useful to grasp the forward legs, one in each hand, and allow the back leg to pass the right thigh with its shoe trailing on the ground, and, thus held, move the instrument bodily about, keeping the eye on the plumb-bob and on the plate levels. Then, before pressing the points of the tripod into the ground, move them slightly in or out or laterally as required to bring the plumb-bob almost over the exact point. In doing this it has to be remembered that a motion of a tripod shoe outwards or inwards in the direction of the leg causes a corresponding motion of the plumb-bob (but only of about half the amount) in the same direction, and on fairly level ground does not much alter the inclination of the parallel plates, while the swinging of a shoe in an arc of a circle about the plumb-bob

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as centre alters the inclination of the parallel plates, but hardly shifts the plumb-bob. Only with practice can the surveyor become adept at swinging the legs and moving them simultaneously in or out, so as to arrive at accurate centering, keeping the parallel plates at the same time nearly level, in the shortest possible time. The final adjustment of the plumb-bob over the station when the theodolite is set up on the ordinary earth surface is effected in pressing the shoes into the ground. The shoes should be pressed in by walking round the theodolite and taking the legs one after the other with both hands and applying a force directly along the leg towards its point. The application of any force tending to bend the legs should be avoided, and the surveyor should never attempt to push in a leg while standing at the other side of the instrument.

On steeply sloping ground the setting up will be most readily and quickly accomplished if two legs are placed downhill and one uphill. If this precaution is not observed it will be found very difficult to get the head on the instrument level.

On pavements and hard, smooth surfaces precautions require to be taken to prevent the shoes from slipping. They should be placed, where possible, in cracks or joints, and it may sometimes be necessary to make a nick for the shoe with the point of a pole or with hammer and chisel. Looseness of the joints at the tripod head produces the worst effects when the theodolite is set up on a hard surface.

It is sometimes necessary, especially in the setting out of works, to measure with chain or tape to a station while the theodolite is set up over it. It is then important to see that the instrument is so placed that none of the legs obstruct the chain line.

If the theodolite is provided with a shifting head the final adjustment over the station is much more rapidly effected. Get the plumb-bob to within about $\frac{3}{4}$ in. of the exact point, with the legs firmly planted and the parallel plates sufficiently level, and then move the plumb-bob the rest of the distance by means of the shifting head.

The more carefully the theodolite is set up in the first instance the less time will be expended in levelling up by the thumb-screws and the less will be the wear and tear of the instrument. The knack of rapid and accurate setting up should be sedulously

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cultivated by the aspiring surveyor, and can only be acquired by practice.

Levelling Up.—To level up the theodolite start with the lower clamp or with both clamps loose. Turn the upper portion of the instrument so as to bring each vernier plate level parallel to a pair of diagonally opposite levelling screws where there are four screws. Bring the bubble of one of the levels to the centre of its run by turning the pair of screws parallel to it at a uniform rate in opposite directions. See that both screws remain bearing on the lower plate without becoming tight. Bring the bubble of the second level to the centre of its run in the same way by turning the second pair of screws. This will probably disturb the first level somewhat, and the levelling of each must be repeated till both bubbles are central; they should then remain central when the instrument is rotated. If, as is generally the case, one of the levels is more sensitive than the other, it should be used exclusively for the final levelling. Turn it first parallel to one diagonal pair of screws and then parallel to the other, levelling it each time until it remains level when the instrument is rotated.

When there are only three levelling screws, turn one of the levels parallel to a pair of screws and bring its bubble to the centre of its run by turning these screws. Bring the bubble of the other level to the centre of its run by turning the remaining screw and repeat till both bubbles are central.

It is not advisable to spend time in bringing the bubble of the first level exactly to the centre of its run. Get both bubbles rapidly to an approximately central position and then proceed to level them exactly.

The levelling screws must be handled very carefully. If a screw begins to work tight it should never be forced. In the four-screw arrangement the stiffness of one pair of screws is sometimes relieved by slacking back both of the other pair of screws a little. When the tightness is due to the instrument being set up very much off the level, the only remedy is to plant afresh.

Measuring Horizontal Angles.—It is required to read the horizontal angle ABC (Fig. 135) with the theodolite, which has been set up and levelled at point B. The upper and lower clamps are loose. Turn the vernier plate and graduated circle relative to each other

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till the index of vernier A is nearly opposite the zero of the circle. Set the upper clamp, thus fixing the vernier plate and graduated circle together, then turn the upper tangent screw till the index coincides exactly with the zero. For the final setting, to avoid parallax, make sure that the centre of the reading glass is vertically over the index, and see that the lines to each side of the index are symmetrical with respect to the circle divisions.

Now apply the fingers of both hands to the edge of the graduated circle and turn the head of the instrument till the telescope is pointing nearly to A, as judged by sighting with the eye along the top of the telescope. Then look through the telescope, focus it on the sighting mark at A, and adjust it by hand so that the cross hairs are as nearly as possible on the mark in both the vertical and horizontal direction. Then fix the lower clamp and bring the cross hairs horizontally into coincidence with the mark by turning the lower tangent screw. If necessary, fix the vertical circle clamp and use its tangent screw for the vertical adjustment. The telescope is now pointing to object A, the upper and lower clamps are fixed, and the vernier indicates zero. To read the angle between the lines BA and BC, the lower clamp therefore remains set, the upper clamp is loosened and the telescope is turned in a clockwise direction and pointed towards C by applying the fingers of both hands to the edge of the vernier plate or the foot of the standards. Set the centre of the cross hairs exactly on point C by fixing the vernier plate clamp and using its tangent screw. The angle ABC can now be read off from vernier A.

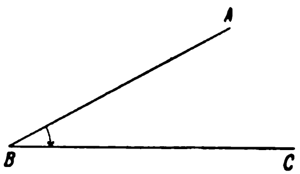

FIG. 135.—Measuring a Horizontal Angle.

It is not essential that the vernier should first be set to zero in order to read a horizontal angle. This is done for convenience and in order to get a direct reading of the angle. The telescope may be pointed to A with the vernier set at random. Read off the angle indicated by the vernier and with lower clamp fixed, upper clamp loose, turn the telescope on to C and again read the angle. The difference of these angles gives the angle ABC, but if the vernier in passing from the first to the second position has crossed the zero of the circle, 360° must be added to the second angle before the subtraction is made.

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In reading a series of adjacent angles from the same point the vernier would, in general, be set to zero for the first sight. Then, without shifting the lower clamp, the telescope would be turned in succession towards the other marks and the angles read off from the vernier. The angles so read would be angles measured round in a clockwise direction from the line of the first sight. The actual angle between any two adjacent directions would be got by subtraction.

If the bearing of line BA is known with respect to some reference direction, such as magnetic or true north, and it is required to find the bearing of line BC with respect to the same direction, the telescope should be pointed to A with the vernier set to indicate the bearing of BA. The angle read off when the telescope is pointed to C will then be the bearing of BC.

Measuring a Vertical Angle.—(*a*) Angle of elevation or depression from a horizontal plane. The theodolite having been planted and levelled up, fix the vernier plate clamp and leave the lower clamp loose. Bring the telescope nearly to the level, fix the upper circle clamp, and by turning the tangent screw bring the zero of the upper circle to exact coincidence with the vernier index. The telescope bubble should be nearly level. Bring it exactly to the level by turning the opposing screws which hold the clipping arm to the attachment on the standard. This will cause the rotation together of the index arms, vertical circle, and telescope, so that when the telescope is level the indexes are at zero and the instrument is in readiness for reading an angle of elevation or depression. Then, to read such an angle, loosen the vertical circle clamp, sight the telescope on the object, clamp the vertical circle and adjust the horizontal cross hair exactly on to the point by turning the tangent screw. The required angle is then given on the vertical circle.

The adjustment to make the telescope bubble remain exactly central as the head of the instrument is rotated is described in Chapter XXII.

Instead of adjusting the vernier to read zero when the telescope is level its actual reading may be noted and treated as an index error. Any angle read off the vertical circle would then require to be corrected by adding or subtracting the amount of this error according as it occurred on the opposite or the same side of the zero.

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(*b*) Vertical angle between two points. To measure the vertical angle between two points sight the telescope on one of the points, bring the horizontal cross hair exactly to the mark, and read the angle on the vertical circle. Then sight to the second point and again read the angle. The difference or sum of these angles will give the required vertical angle according as they occur on the same or on opposite sides of the zero.

Measuring Angles by Repetition.—By repeating the measurement of an angle several times in such a way as to add up the successive readings on the graduated circle a more accurate determination of the angle can be made than by a single reading. The method is described in Chapter XIV.

Doubling the angle forms a useful check to ensure that no serious mistake has been made in the reading. The angle is measured once and its value booked. The angle is then repeated and the circle reading is noted and divided by two. This should give a value almost the same as the first reading, and any serious error would be at once apparent.

Ranging a Straight Line.—With the aid of a theodolite, survey lines can be ranged out with almost perfect accuracy, and straight lines can be set out across hills and hollows with little trouble. To set out points in line between two survey stations for the guidance of the chainmen the theodolite will be set up and levelled over one of the stations. Particular attention should be paid to the vertical adjustment of the instrument, and to the level which is parallel to the horizontal axis of the telescope. With the vernier plate clamp fixed and the lower clamp loose, sight the telescope on to the bottom of a pole held at the distant station. Fix the lower clamp and use the lower tangent screw to bring the centre of the cross hairs exactly on to the pole. Both upper and lower clamps are now fixed and must remain so while the points are being lined in. The telescope can turn vertically on its horizontal axis so that its line of sight moves in a vertical plane passing through the two stations, and any point where the line of sight strikes the ground will lie on a straight line between the stations. Poles should be planted in order coming from the distant station towards the theodolite, so that when set they may not obstruct the view of the next point. In directing the assistant into line the observer

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at the theodolite looks first along the top of the telescope and signals the assistant to move in the required direction. When he comes into the field of view the observer looks through the telescope, gets the pole into focus, and, sighting as near the ground as possible, directs the assistant to move the pole laterally till it is bisected by the centre of the cross hairs. He then gives the signal to plant.

In ordinary surveying work there is no need for great refinement in setting the poles in line. The thickness of a pole will hardly affect the accuracy of the survey. In town work, however, and in the setting out of works, accurate alignment is essential, and the theodolite should be sighted on a fine mark, such as is afforded by the point of a pencil or arrow or the string of a plumb-bob.

To fix an intermediate survey station in line between two main stations the peg to be driven should, where practicable, be sighted to instead of a pole. The observer at the theodolite should watch the peg as it goes down and direct the driving so that the middle of

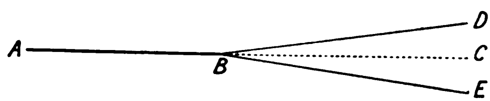

Fig. 136.—Prolonging a Line.

the peg is kept nearly in line. If accurate lining is necessary, or if the distance between pegs is short, a small nail should be lined in on the head of the peg and driven to mark the exact point. When the position of the peg is invisible from the theodolite owing to long grass or otherwise a ranging pole or the string of a plumb-bob should be lined in and the position of the station marked on the ground. The peg may then be driven and the plumb-bob used to test the accuracy of its position and to determine the point for the nail if required.

Prolonging a Line.—It is required to prolong the line AB (Fig. 136) on to point C. If the distance AB is not too great and C is visible from A the theodolite may be set up at A, sighted on to B, and then points lined in ahead up to C.

If the conditions of view require it, the theodolite should be set up at point B, the telescope sighted on to A and then transited so as to point ahead. If the instrument is in correct adjustment, any points which are now lined in will lie on the line AB produced. In

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prolonging a line by the above method errors of adjustment can be eliminated by proceeding as follows: Sight on A with the telescope in its normal position, transit the telescope and set out a point ahead. This point may fall at D (Fig. 136) a little to one side of the correct point C. Now sight on to A with the telescope in its inverted position and again transit the telescope. The same amount of error will be introduced as in the first case, but it will lie to the opposite hand, so that if a point E is now lined out the true point C will lie midway between D and E. The distance DE may be measured and point C fixed at the half distance. The theodolite may then be sighted on to C and intermediate points lined in between B and C.

Another method of prolonging a line AB where both A and B are visible from C is to set up the theodolite as nearly at the point C as can be judged by the eye, sight on to point B, and note whether the centre of the cross hairs also strikes point A. If it does the theodolite is on the correct line. If it does not the theodolite must be shifted by trial till it is found to be in line.

Ranging a Straight Line between Two Stations, neither of which is Visible from the Other.—This corresponds to the case described on p. 30, under chain surveying, where rising ground intervenes between the stations. It is assumed that both stations can be seen from some intermediate point. First employ the method of p. 30 to line in two poles approximately by the eye. Set up the theodolite on the line thus found, sight to one of the stations, and transit the telescope. The line of sight should now strike somewhere near the other station. Note the amount of the deviation and estimate how far the theodolite should be shifted laterally in order to correct it. Shift the theodolite this estimated amount and test as before by sighting on one station and transiting the telescope. A few trials may be necessary before the theodolite is found to be in line. For accurate lining the final test should be made with the telescope first in the normal position and then in the inverted position. The theodolite having been brought into line, intermediate points may now be ranged in.

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CHAPTER XI

TRAVERSE SURVEYING WITH THE THEODOLITE

THIS chapter deals principally with the problems connected with the laying out of a system of traverse survey lines, the fixing of the survey stations, and the reading of the angles. The latter is an important part of the work involved in traverse surveying, and is dealt with somewhat fully. The various methods of procedure in use for measuring the direct angles between survey lines and for finding the whole circle bearings of survey lines are considered, and attention is given to desirable methods of booking angles and bearings.

Traverse Surveying with the Theodolite.—In a traverse survey a system of connected survey lines is laid out from which the objects and natural features are located. The relative directions of the lines are fixed by reading the angles which they make with each other, or by taking their compass bearings.

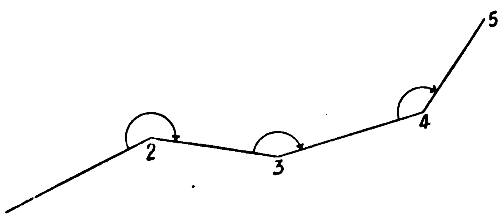

FIG. 137.—Unclosed Traverse.

The arrangement of the lines is not limited to any particular geometrical form as in chain surveying, where a system of triangles forms the fundamental basis of the arrangement. The use and limitations of the surveyor's compass for reading angles and bearings have been described in Chapter IX.

Systems of traverse survey lines may be divided into the three following classes :—

(*a*) Unclosed traverse.

(*b*) Single closed traverse.

(*c*) Network, consisting of combinations of (*a*) and (*b*).

Fig. 137 represents an arrangement of survey lines forming an

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unclosed traverse. It corresponds to the case of a survey for a stretch of road where each successive straight portion is located from a single survey line. The directions of the lines are fixed by reading the angles at the bends.

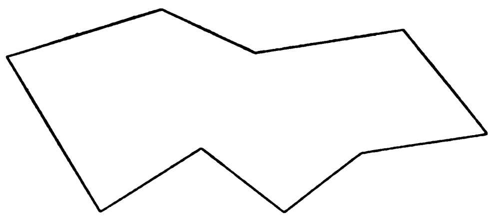

FIG. 138.—Single Closed Traverse.

A single closed traverse is shown in Fig. 138. The lines may form a polygon of any shape and with any number of sides. The survey lines to locate the boundaries of any single enclosure will usually take this form.

The lines to survey any extensive area will usually take the form of a network, composed of a connected series of polygons with or without unclosed branches. This arrangement is typified in Fig. 139.

Laying out Traverse Survey Lines.—The principles which should govern the lay-out of a system of traverse survey lines are in many respects similar to those which control the arrangement in a chain survey. The main difference is due to the fact that in traverse surveying the arrangement is not restricted to a particular form, and in consequence the lines can be laid out to much better advantage as regards ease of location of objects. Time spent in examining the ground to ensure that no serious obstacles to chaining occur on the survey lines will rarely be wasted. The aim should be to have a system which will be economical, both as regards field work and plotting, and to this end the method and requirements of plotting must be kept in view. For plotting purposes generally the fewer the number of lines the better, but

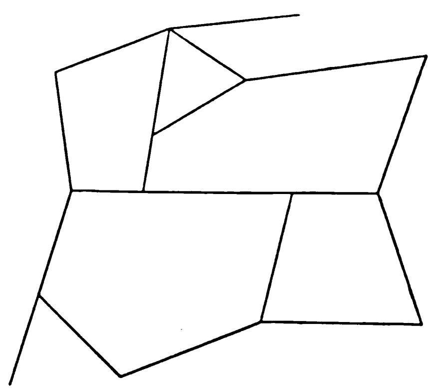

FIG. 139.—Traverse Network.

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this does not always hold good. An extra survey line may sometimes be of great use in simplifying the booking and enabling complicated objects to be more easily and accurately located.

Fixing Survey Stations.—While the survey stations are largely controlled by the conditions considered in the preceding paragraph, they must at the same time be chosen with due regard to the safety and ease and accuracy of manipulation of the theodolite.

In open country firm, level ground for setting up on is to be desired. A position should be chosen where there is little likelihood of the peg being disturbed or removed, and near some permanent and easily recognised object to which it may be referred for easy recovery.

In fixing stations on roads, streets or railways it is important to choose positions where the instrument will be safe and where the reading of angles can be carried on without interruption due to traffic. The setting up of the instrument near the centre of a road or street should be avoided. In streets it will generally be found most suitable to fix the stations on or near the kerb and have the lines run along the pavement or side of the road. In busy roads and streets an endeavour should be made to avoid positions where delay is likely to be caused through traffic.

In surveying railway lines the stations should, as far as possible, be fixed quite clear of the running tracks, so that the observer and instrument will not be in danger from passing trains.

Methods of Reading Angles.—With a theodolite whose circle is graduated from 0° to 360°, either of the following two methods of reading angles in a traverse survey may be employed :—

(*a*) Direct angle or separate angle method.

(*b*) Whole circle bearing method.

If the theodolite circle is graduated according to the quadrant system, the deflection angle method of reading angles may be employed. The quadrant method of graduation is rarely used in Britain.

Direct Angle or Separate Angle Method.—The direct angle method consists in obtaining the actual angle at the junction at each pair of lines. The angle read may either be the interior or the exterior angle, as shown in Fig. 140. In reading a series of angles, however,

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a systematic method of procedure should be adopted. Interior and exterior angles should not be read indiscriminately.

In an unclosed traverse, as illustrated in Fig. 137, assuming that the stations are occupied in the order of their numbers, the method of procedure would be as follows: The theodolite would be set up at station 2 and sighted on station 1, with the vernier set at zero. If the theodolite is provided with a compass the bearing of line 2—1 should be noted as an approximate method of determining true north. The upper clamp would then be loosened and the telescope rotated clockwise and sighted on station 3. The angle which would be read off the circle is shown by the arrow in the figure. On setting up at any other station the telescope would be sighted back on the previous station with the vernier set at zero and then turned on to the forward station. Proceeding in this manner, the angles read would all lie on the same side of the traverse, the calculation of the bearings or azimuth angles of the lines being thereby rendered simple. Angles read by the direct method may be checked by doubling, while, if a more precise value is desired than can be got from a single observation, the method of repetition may be employed. See Chapter XIV.

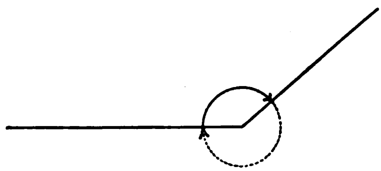

FIG. 140.—Reading Exterior or Interior Angles.

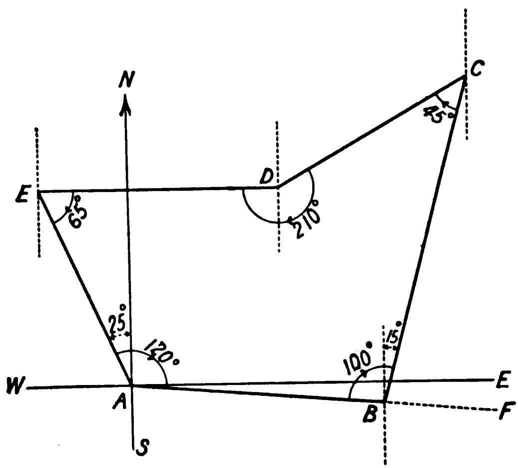

FIG. 141.—Calculation of Bearings from Separate Angles.

Calculation of Bearings from Direct Angles.—In a closed traverse it is generally preferable to read the internal angles. Fig. 141

shows a closed traverse in which the angles read are indicated by small arcs with arrow heads showing the direction of rotation of the theodolite. The observed compass bearing of the line AE is 25° west of magnetic north and the angle EAB is 120°. The whole circle bearing of the line AB is therefore 120° — 25° = 95°. Consider now the angles at the point B. The bearing of the line BF, which is AB produced, is 95°. A pointer hinged at B and pointed in this direction would require to be rotated in a clockwise direction through 180° + 100° to bring it round to the direction BC. The whole circle bearing of BC is, therefore, 95° + 180° + 100° = 375°. As this value is greater than a complete revolution, 360° must be deducted from it, so that the proper bearing is 375° — 360° = 15°. In the same way the bearing of CD is got by adding 180° + 45° to the bearing of BC, and so on for the other lines. The bearings worked out for the whole polygon are as shown in the following table :—

Line.	Bearing.
AB	120° — 25° = 95°
BC	95° + 180° + 100° = 375° = 15°
CD	15° + 180° + 45° = 240° = 240°
DE	240° + 180° + 210° = 630° = 270°
EA	270° + 180° + 65° = 515° = 155°

In reading angles by the direct angle or separate angle method, as above described, each angle is dealt with independently of any other. The angles may, therefore, be read in any order, and this is sometimes an advantage as compared with the whole circle bearing method to be next described.

When all the interior angles of a polygon have been read a check on their accuracy is obtained by adding them together and noting whether their sum amounts to an even number of right angles. In a polygon of N sides the sum of the interior angles should be equal to 2N — 4 right angles, or equal to 180 (N — 2)°.

If the angles are read in sequence round the polygon, and the bearing of each line is calculated as the work proceeds, a rough check will be afforded by comparing the calculated bearing with the compass bearing.

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Whole Circle Bearing Method.—In this method, known also as the azimuth angle method, a reference direction, sometimes known as a reference meridian, is chosen, and the direction of any given line is determined by measuring the clockwise angle between this reference direction and the given line. The angle will lie between 0° and 360°. The reference meridian is generally taken as magnetic north or true north, but any other direction, such as that of a particular survey line, may be adopted if more convenient.

The three following methods of procedure are used for the purpose of finding consecutively the whole circle bearings of the lines of a traverse survey :—

(*a*) Direct bearing method.

(*b*) Back bearing method.

(*c*) Method by ignoring distinction between forward bearing and back bearing.

Method (*a*).—The procedure applied to the determining of the whole circle bearings of the two lines AB and BC by the direct bearing method is illustrated in Fig. 142. Magnetic north is assumed to be the reference direction. The instrument is set up and levelled at station A. The vernier of the horizontal circle is set to zero and clamped, the lower clamp being meantime loose. The magnetic needle having been lowered on to its pivot, the plates and telescope are rotated together till the zero of the compass circle is brought to coincide with the north-pointing end of the needle. The telescope then points in the same direction as the needle, that is, towards magnetic north, and the index of the horizontal circle is still at zero, or, in other words, the telescope now points in the reference direction and the reading is 0°. The lower clamp is now set, thus fixing the horizontal circle, and the upper clamp is loosened. The telescope is then rotated clockwise and directed on to station B and the angle is read off. This angle is the whole circle bearing of the line AB with reference to magnetic north. In the small diagram (A) (Fig. 142) the full lines represent the telescope pointing to magnetic north while the circle reading is zero. The dotted lines represent the position of the telescope when pointing towards B. The lower clamp is loosened, and the instrument may then be lifted and transported to station B and set up, the plates remaining clamped at the bearing of AB. The

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instrument having been set up and levelled, the telescope is sighted back on station A, the lower clamp is fixed, and the cross hairs are accurately set on A by turning the lower tangent screw. The position of the telescope and horizontal circle is now as shown in full lines on small diagram (B) (Fig. 142). It will be noted that the diameter of the circle which pointed to magnetic north at station A points in the same direction at station B, but turned through 180°. The telescope is now transited, so that if it was normal at A it will now be inverted, and the vernier will still be reading the bearing

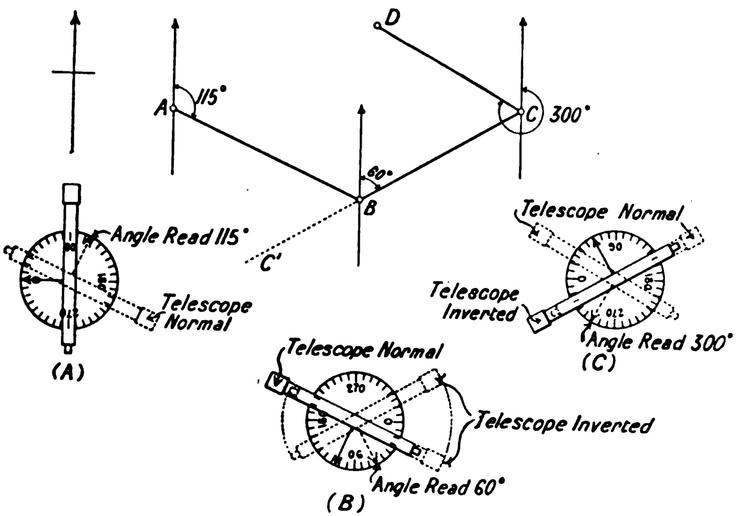

FIG. 142.—Whole Circle Bearings by Direct Bearing Method.

of line AB. The transiting of the telescope cancels the effect of the rotation through 180° of the horizontal circle which has taken place. Now loosen the upper clamp and sight the telescope on to station C and read the angle, which will be the forward whole circle bearing of the line BC.

The procedure at any other station such as C is similar to that at B, and may be summarised as follows :—

(*a*) Set up at C with vernier clamped to bearing of BC and the telescope in the position used for reading that bearing. Lower clamp loose.

(*b*) Sight telescope back on station B, using lower clamp and tangent screw for accurate adjustment.

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(*c*) Transit the telescope, loosen the upper clamp, and sight the telescope on station D, using the upper clamp and tangent screw for accurate setting.

(*d*) Read off the angle which will be the whole circle bearing of the line CD.

A reference to Diagrams (A), (B) and (C) (Fig. 142), should make the procedure clear and show how the angles read are the whole circle bearings of the various lines. If the theodolite has more than one vernier the same vernier must be used throughout.

The method of procedure above described, in which each angle is read only once, furnishes in itself no check on the accuracy of the angles. Comparison of the bearings with the readings of the compass (if there is one) will give a very rough check. To obtain greater reliability proceed as follows at station B after having read the bearing of the line BC: Loosen the upper clamp, rotate the telescope clockwise and sight back on to station A. The circle reading should now be equal to the forward bearing of the line AB increased or diminished by 180°. If there is no discrepancy, rotate the telescope clockwise and again direct it on to station C, and read the bearing. If this is the same as before, the angle may be accepted as correct. The upper clamp would, therefore, be set at this reading, the lower clamp would then be loosened and the theodolite could be transported to the next station.

If on sighting back on to station A a small discrepancy is found in the bearing, it will be necessary to repeat the operation.

Where the bearings of several lines require to be read from one station, read first the bearings of all the lines, taking them in order in a clockwise direction, and then check back on to the starting station. Finally, sight the telescope on to the station at which the next set-up is to take place, see that the bearing agrees with the former reading, clamp the plates at this bearing and carry the instrument forward.

Method (*b*).—In the back bearing method of procedure the telescope is always kept in its normal position ; it is never transited. Having set up the instrument at B set the vernier to the whole circle bearing of BA, which is the forward bearing of AB increased by 180°. Sight on to station A and then, with the lower clamp fixed, rotate the telescope clockwise and sight on station C. The

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reading will be the whole circle bearing of the line BC. To ensure accuracy check back on to the starting station and repeat the observation.

Method (*c*).—In this method the distinction between forward bearing and back bearing is ignored. On setting up at B the telescope is sighted on to station A with the vernier set to the *forward* bearing of line AB. The bearing determined for BC will, therefore, be wrong by 180°. This will, however, be the correct backward bearing of CB, so that on setting up at C and proceeding as before the bearing of CD will be correctly determined. The bearings of alternate lines will in fact be correct while the others will be wrong by 180°. The only mistake in plotting which can arise thereby is the laying off of one survey line to the wrong hand with respect to another, as in the direction BC′ instead of BC (Fig. 142). Such a mistake will not occur if a sketch has been made showing the survey lines in fair relationship to each other, and in any case would not remain undetected in a closed traverse.

Comparison of Methods of Measuring Whole Circle Bearings.—In the first or direct bearing method the vernier clamp may slip, due to jolting in carrying the instrument from one station to another, and hence the circle reading should be checked after the instrument is set up. If the instrument is not in correct adjustment in respect that the line of sight of the telescope is not exactly perpendicular to its horizontal axis, or if its horizontal axis is not in correct adjustment, an error will be introduced at each angle, due to transiting the telescope. This source of error is avoided in methods (*b*) and (*c*). Method (*b*), or the back bearing method, has the disadvantage that the back bearings of the lines require to be calculated and that the vernier requires to be set to a particular reading at each station. Both operations afford opportunity for the introduction of error and take up time. Method (*c*) avoids the necessity for calculation of bearings, but the vernier is liable to slip.

Booking Angles of Traverse Survey.—Where the arrangement of survey lines is not complicated the most convenient method of recording the angles is to figure them directly on a sketch of the lines. A fair sketch of the lines with all stations lettered or numbered and with the length and bearing of each line plainly figured alongside is

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a necessity if the survey lines are to be expeditiously plotted. Wherever the method of taking the bearings involves calculation and where the sketch method is not convenient, the following tabular system of booking the angles is recommended :—

Theodolite Station.	Station observed.
A	B = 335° 40′
B	A = 155° 40′ L = 191° 13′ C = 250° 4′ K = 63° 35′
C	B = 70° 4′ D = 293° 15′ M = 342° 45′
D	C = 113° 15′ E = 197° 20′

The above system keeps a record of the back sight and its bearing taken at each station as well as the bearings of the several lines read from it. The back sight from C to station B is set down as 70° 4′. This is obtained by calculation from the bearing of BC previously found. The figures being set down in the notebook enables a mistake in calculation to be afterwards found out by checking and corrected, while, if the calculation is merely done mentally and not recorded, the detection of an error is a difficult matter.

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CHAPTER XII

PLOTTING A TRAVERSE SURVEY BY ANGLE AND DISTANCE

THERE are two principal methods of plotting the survey lines of a traverse survey, viz., by angle and distance and by co-ordinates. The various ways in which the angle and distance method of plotting a system of traverse survey lines may be applied are considered in this chapter. The plotting of angles and bearings may be undertaken by methods which correspond to the direct angle and whole circle bearing methods of measuring angles, with variations in each case according as the angles are laid off by protractor or by geometrical construction. The methods of laying-off angles and bearings by the protractor and by geometrical constructions, utilising the tangent or chord of the angle, are dealt with, and the advantages of the various methods are compared. Consideration is also given to the methods of checking and ensuring the accuracy of unclosed traverses, and to the adjustment of the closing error in a closed traverse.

The angle and distance methods of plotting traverse surveys described in this chapter are only suitable for small surveys, and are much inferior, in respect of accuracy of plotting, to the co-ordinate method described in the next chapter.

Methods of Plotting.—In laying down the directions of traverse survey lines on paper for the purpose of plotting the plan we may proceed according to either of the two general methods already mentioned, viz. :—

(*a*) Direct or separate angle method.

(*b*) Whole circle bearing method.

(*a*) The direction of each survey line is laid off from the previously plotted line to which it is connected by plotting on the paper the angle at the junction of the lines.

(*b*) The directions of the lines are laid down relative to a fixed reference direction.

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Of the two methods the latter is the more useful and reliable.

Whichever is used the separate angles and bearings may be plotted by protractor or by geometrical construction in several different ways, the following methods being noteworthy :—

(*a*) By protractor.
(*b*) Tangent method.
(*c*) Chord method.

Methods (*a*) and (*b*) are the more useful for purposes of plotting a traverse survey by angle and distance.

Laying off a Single Angle or Bearing.—(*a*) By protractor. To lay off a given angle at point B on the base AB, first produce the line AB onwards by a length rather greater than the radius of the protractor. Then set the 0° and 180° points of the protractor on this line and bring the centre mark to coincide exactly with the point B. Make a prick mark opposite the proper graduation on the circumference and join it up to point B.

The usual form of semicircular protractor of brass or other material is graduated from 0° to 180° in both directions, so that either clockwise or anti-clockwise angles may be set off. The accuracy of the work is limited by the size of the radius, and as this does not commonly exceed 6 ins., such a protractor is only useful for plotting short lines and details.

By using a full-circle protractor and pricking off both the given angle and its supplement, the plotted side of the angle will be fixed by two points a whole diameter apart, and hence greater accuracy will be attainable than when the points are only a radius apart.

A good form of protractor for plotting survey lines is the large circular cardboard type, 18 ins. to 24 ins. in diameter. The circumference is graduated to ten-minute intervals, and these are large enough to permit of single minutes being pricked off by estimation with an error not exceeding two or three minutes. For plotting whole circle bearings there are two sets of graduation figures running both in clockwise direction from 0° to 360°. The one set follows the other with an interval of 180°, so that any given angle occurs twice at points diametrically opposite each other. In using this protractor both points are pricked off, thus giving a base equal to the whole diameter. Directions so pricked off are transferred and made to pass through the required station point

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on the plan by the use of a parallel ruler or other equivalent means. The centre of the protractor does not in general require to be used in setting, or in laying off angles.

(*b*) By tangent methods. In the right-angled triangle (Fig. 143), if the base is one unit long and the length of the perpendicular is t, the tangent of the angle a will be $\frac{t}{1} = t$. Any angle can, therefore, be laid off by erecting on a base line one unit long a perpendicular equal in length to the natural tangent of the angle. If the base is made a certain number of units in length the perpendicular will require to be made equal to the tangent multiplied by that number. For many purposes a convenient length of base is 10 ins., and in that case the height of the perpendicular in inches is ten times the natural tangent.

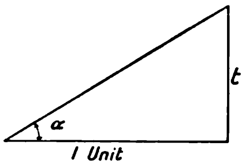

Fig. 143. — Plotting Angle by Tangent Method.

For angles less than 45° the length of the perpendicular is less than that of the base. For angles greater than 45° the perpendicular is greater than the base, and as the angle approaches 90° the perpendicular becomes immensely greater. It is, therefore, not convenient to plot angles which are much greater than 45° directly by the tangent method. For such angles it is better to plot the complement of the angle from a base drawn perpendicular to the given side. In Fig. 144 the plotting of the angle a on the side ab requires the erection of the long perpendicular bc. This is avoided by plotting the angle β from the base ad drawn at right angles to the line ab. If ad is 10 ins. long $de = 10 \tan \beta = 10 \tan (90 - a)$.

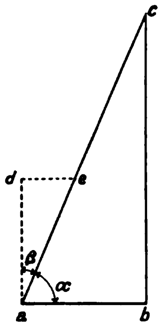

Fig. 144.—Plotting Angles over 45°.

Fig. 145 shows how the direction of any whole circle bearing between 0° and 360° can be laid off by utilising the sides of four squares as bases. A little consideration will show that one square can quite well serve the purpose of the four.

The accuracy of the tangent method of plotting angles depends on the precision with which distances are scaled and right angles laid off.

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Particular care should be taken to see that the set-squares used are accurate. The accuracy of base lines laid out in the form of a square is best tested by scaling the lengths of the diagonals and noting if they are the same.

(*c*) By chord method. It is required to lay off an angle a with

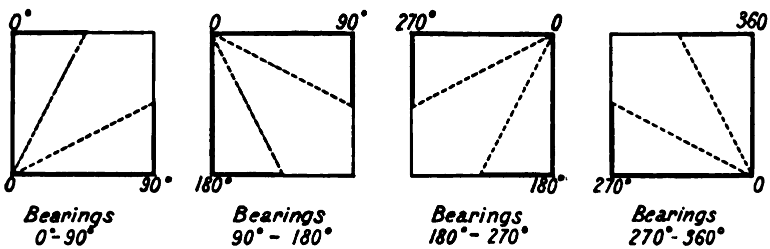

FIG. 145.—Plotting Bearings by Tangent Method.

apex at a on the line ac (Fig. 146). With a as centre and radius ab equal to a convenient number of units, say, ten, an arc bd is struck. From centre b and with radius equal to the chord corresponding to angle a for a radius ab another arc is struck to intersect the former in point d. The required angle dab is got by joining point d to point a. The lengths of chords of angles corresponding to unit radius are given in various sets of mathematical tables. The chord bd corresponding to the angle a and unit radius ab is equal to $2 \sin \frac{a}{2}$.

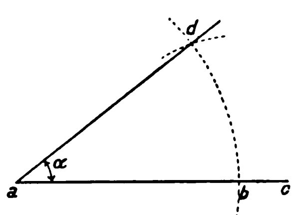

FIG. 146.—Plotting Angle by Chord Method.

Direct or Separate Angle Method of Plotting Traverse Survey Lines.—(*a*) Using the protractor. Fig. 147 illustrates the direct or separate angle method of plotting survey lines by protractor. Before commencing to lay down any lines on the paper, precautions must be taken to ensure that the survey lines when plotted will lie in proper position on the sheet. See Chapter VIII.

In Fig. 147 the line AB is taken as the base. The direction of the line AE is got by placing the protractor as shown, and making a prick mark at the 71° graduation. The point E lies on the line drawn through this prick mark and point A, and its position is

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fixed by scaling off a distance of 707 ft. from point A. Lines BC and CD are laid off successively in a similar manner to the line AE.

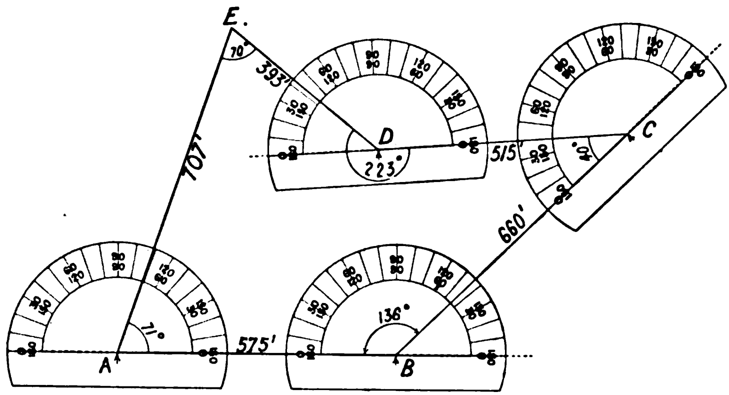

FIG. 147.—Plotting Survey Lines by Protractor.

The angle of 223° at point D is plotted by laying off an angle of 43° above the line CD produced. The last line drawn from D should pass through the point E already plotted, and the scaled length of DE should be the same as the actual length measured on the ground.

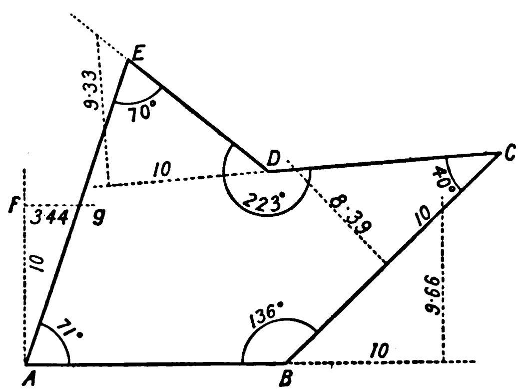

FIG. 148.—Plotting Survey Lines by Tangent Method.

If the final plotted point E′ does not coincide with the first point E the interval EE′ is known as the closing error. The closing error of a traverse plotted with a protractor will, in general, be due to a combination of errors in surveying and errors in plotting. If the closing error turns out large, mistakes should at once be looked for in the plotting. To test for angular errors, check first the angle which the direction of the last line makes with the first line. If this does not agree with the measured angle, check the others till the mistake is located. If the angles are found to be correct the error must be looked for in the scaling.

The distances and angles of an unclosed traverse require very careful checking to ensure that no errors have occurred in the plotting, as the work affords no check in itself, such as is obtained on completing a closed traverse.

(*b*) Tangent method. Fig. 148 illustrates the tangent method of laying off angles applied to the plotting of traverse survey lines successively. The first step is to make out a table showing the angles to be plotted with the values of their tangents and the lengths of the survey lines as follows :—

Angle plotted at	Tangent.	Line.	Length.
A = 90° — 71° = 19°	0·3443	AB	575′
B = 180° — 136° = 44°	0·9657	BC	660′
C = 40°	0·8391	CD	515′
D = 223° — 180° = 43°	0·9325	DE	393′
E = 90° — 70° = 20°	0·3640	EA	707′

The line AE, making an angle of 71° with the base AB, is got by laying off an angle of 19° on the right-hand side of a line drawn up at right angles to AB from the point A. The angle of 19° is plotted by marking off the length A*f* equal to ten units and from *f* constructing the offset *fg* equal to ten times the tangent of 19° or equal to 3·443 units. The units on any engineer's decimally-divided scale may be used instead of inches if a length of base other than 10 ins. is thought desirable. The line A*g* makes the required angle with the line AB, and the point E is plotted by scaling off a length of 707 ft. from A. The direction of the line BC is got by marking off from B a base of ten units along the line AB produced, and erecting an offset of 9·66 units at its right extremity. The lines CD and DE are plotted successively in the manner shown in the figure. If the traverse does not close, the angle made by the last line drawn from D with the line AE should be checked. If no error is found in the angle the lengths of the traverse lines must be checked.

Whole Circle Bearing Method of Plotting Traverse Survey Lines.—(*a*) *By protractor.* This method is illustrated in Fig. 149, and with

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careful draughtsmanship and the use of a large and accurate protractor enables traverse survey lines to be laid down with fair accuracy within, at least, the limits of a double-elephant sheet. A line parallel to the base AB, *fg* in the figure, is drawn in a convenient central position on the sheet and the protractor is placed over this line so that the graduations representing the bearing of AB are coincident with it. The protractor is thereby set with its zero and 180° points on the reference meridian, and the bearings of the other lines may be pricked off round its circumference and

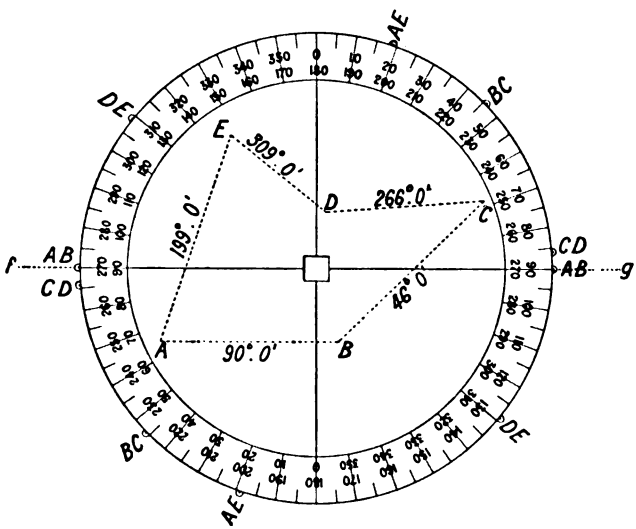

FIG. 149.—Plotting Whole Circle Bearings by Protractor.

marked for identification in the manner shown. Instead of marking the bearing by the line to which it refers, its value may be written on the paper at each prick mark. To lay off the direction AE through point A, set a parallel ruler with its edge passing through the two prick marks designated AE. Then roll it till the edge passes through point A and draw a fine pencil line through it in the proper direction. Point E is plotted by scaling along this line from A. The other lines are similarly laid off by transferring the directions with the parallel ruler, and the accuracy of the work is proved if the last line drawn through D passes through point E and scales correctly. This method of

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plotting requires that the parallel ruler should be very carefully used to ensure accurate results. It should be carefully tested to see that it rolls true by the method described on p. 87. A good check against mistakes in marking off the bearings is got by pricking the centre point of the protractor on to the paper and, as the edge of the ruler is laid across each pair of points, noting whether it also passes through the centre point. A mistake in marking off one of the points from the protractor will thereby be at once detected, as the line joining the points would not then pass through the centre.

As compared with the two previous methods of plotting traverses the whole circle bearing method has the advantage that one setting of the protractor serves to lay down all the bearings. Also, while in the previous methods a small error in the direction of any one line affects the direction of all subsequent lines connected thereto, in the whole circle bearing method each angle is plotted independently of any other, and a small error in the bearing of one line does not affect the direction of any other line. The method has the further advantage that the full diameter of the protractor is utilised in laying off the angular directions.

(*b*) By tangent method. This method of plotting whole circle bearings is illustrated in Fig. 150. The bearings of the survey lines require first to be reduced to the angles less than 45° which they make with the meridian direction or with a direction at right angles to the meridian. The meridian direction is represented by the bearings 0° and 180°, and the other direction by the bearings 90° and 270°. The following table shows the reduced angles for the given traverse, and the tangents required in plotting :—

Line.	Length.	Bearing.	Angle Plotted.	Tangent.
AB	575	90° 0′	0°	·0000
BC	660	46° 0′	90° — 46° = 44°	·9657
CD	515	266° 0′	270° — 266° = 4°	·0699
DE	393	309° 0′	309° — 270° = 39°	·8098
EA	707	199° 0′	199° — 180° = 19°	·3443

A square is constructed near the centre of the sheet of paper, having sides long enough to form suitable bases for the plotting of the angles. Sides 10 or 20 ins. long may be used, according to

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the length of the survey lines. The square will not necessarily be placed with its sides parallel to the edges of the paper, but must be arranged to suit the desired disposition of the survey lines. Having fixed one of the survey lines on the paper, plot from this line by the tangent method either the meridian direction or the direction perpendicular to it and make the sides of the square parallel to these. Whole circle bearings between 0° and 45° will be plotted by making *gf* the base and laying off the offsets along *fk*. For bearings between 45° and 90° *gh* is the base and offsets are

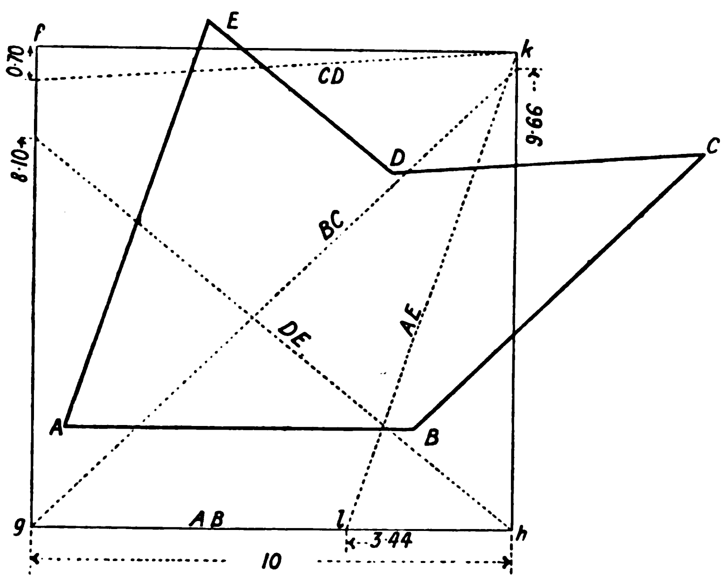

FIG. 150.—Plotting Whole Circle Bearings by Tangent Method.

erected along *hk*. For bearings between 90° and 135° *fk* is the base and offsets are plotted down the line *kh*, and so on. In plotting the bearing of line EA, which is 199°, the line *kh* represents a bearing of 180° and the additional angle, namely, 19°, is plotted on *hg* as base. If *kh* is 10 ins. long the offset *hl* will be 3·443 ins. long, and the line *kl* will give the direction of AE. The directions of the other lines are laid off in similar manner. The plotting of the survey lines is effected by transferring these directions by parallel ruler or other means, the procedure being the same as when the angles are plotted by protractor.

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Comparison of Angle and Distance-Methods.—The direct angle method of plotting survey lines, in which the direction of each line is laid off relative to that of the previously plotted adjoining line, is not recommended except for rough purposes. Angular error occurring at any point is continued through the rest of the work, and the method is laborious in respect of the large number of separate settings of the protractor, or of the separate geometrical constructions required in order to lay off the directions of the various lines.

Where the angles of a traverse have been measured by the direct angle method, the preferable procedure is to reduce them by calculation to whole circle bearings in the manner shown on p. 147, and then adopt one of the corresponding methods of plotting.

Of the angle and distance methods of plotting a traverse, the whole circle bearing method using a large and accurate protractor is the most generally useful. The tangent method, with the use of base lines of suitable length, will give as accurate results as the protractor method, but it is somewhat more laborious and affords more opportunity for mistakes occurring in the reduction of bearings and abstracting of trigonometrical values from the tables. It is, at the same time, a most useful method to have at command should the occasion arise when a protractor is not available.

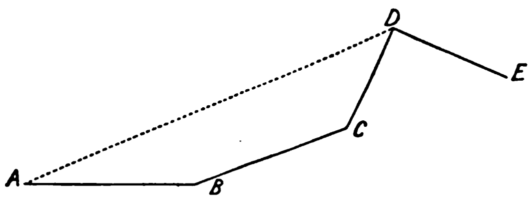

FIG. 151.—Method of Checking an Unclosed Traverse.

Checks on Unclosed Traverse.—A method of ensuring accuracy in the bearings of an unclosed traverse is illustrated in Fig. 151. From station A, in addition to reading the bearing of the first line AB, take also, if reasonably possible, the bearings to one or more of the forward stations, such as D. On leaving station A the theodolite would be set up successively at B and C to get the bearings of the lines BC and CD. On setting up at D the back sight should be taken on to station A, as giving a longer sight and a more directly obtained bearing than station C, and, after taking

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the bearing of DE and check bearings where possible to stations ahead, read the bearing of DC. If this corresponds with the bearing of the same line taken from C, the angular work up to station D may be accepted as correct. If there is a small discrepancy not exceeding the unavoidable error for the instrument used, the bearing of DC taken from station D will be accepted as correct, the bearings of BC and CD being adjusted slightly to get rid of the angular closing error. By proceeding in this way the surveyor will, under favourable conditions, obtain a check on the angular work every few stations and will be enabled to leave the field confident of its accuracy.

By making use of the observed check bearings in the plotting a partial check on the accuracy of the linear work may also be obtained. If the bearing of AD drawn through A is found to pass

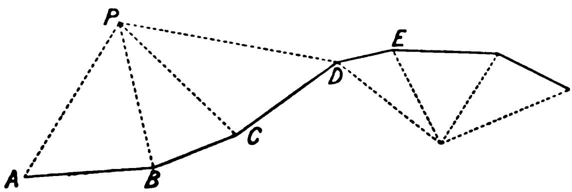

FIG. 152.—Checking Unclosed Traverse by Bearings to Lateral Object.

through point D, the lengths may be presumed to have been correctly taken and plotted as well as the angles. An error in measuring or plotting the length of the lines AB or CD would evidently cause point D to deviate from the line of the bearing drawn through A. If, however, any line, such as BC, had the same, or nearly the same, bearing as AD, an error in its plotted length due to measurement or scaling would not cause point D to deviate from the line AD, and hence would not be detected.

A method which furnishes a check when the work is plotted consists in reading the bearings to a conspicuous side object from each of three or more consecutive stations, the whole traverse being dealt with by a series of such groups of stations. In Fig. 152 side bearings have been taken from stations A, B, C, and D to the point P. The check in plotting consists in laying off these bearings through their respective stations on the paper and noting whether the lines pass through one point. If they do, the work is presumably

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accurate. The method is not a very satisfactory one. The angles are not checked in the field and the office check is often awkward owing to the intersection points of the side bearings falling off the paper.

For important work the most satisfactory method of checking the linear measurements consists in chaining each survey line a second time, preferably in the reverse direction.

Graphical Adjustment of Closing Error.—A graphical method of adjusting the unavoidable closing error in a traverse plotted by any of the foregoing methods is illustrated in Fig. 153. The

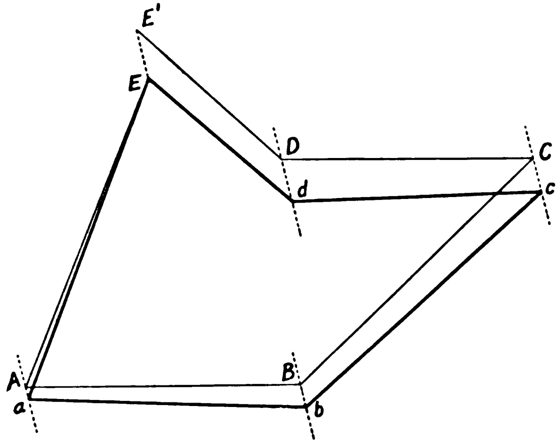

FIG. 153.—Graphical Adjustment of Closing Error.

starting point of the traverse is E and the last line drawn from D terminates at E′, giving a closing error EE′. To eliminate the closing error E′ must be moved to E, and, to avoid making a large adjustment in any single line, each of the other station points should be moved a certain distance parallel to E′E, the amount of the shift for each point being proportional to its total distance from the starting point E. The amount of the shift for the various station points is obtained graphically, as shown in Fig. 154. The lengths of the survey lines starting from point E are laid off to a convenient scale along the line EABCDE′, and the perpendicular E′E″ is constructed equal in length to the closing error on the paper. The line joining E to E″ will by the cut-off of the perpendiculars

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erected at A, B, C, and D give the required amounts of the shifts at these stations. The various stations being shifted parallel to the direction EE′, as shown in Fig. 153, by the amounts thus

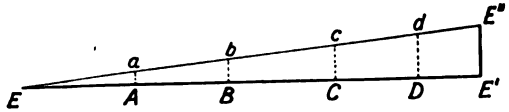

FIG. 154.—Distribution of Error in Traverse.

obtained we get the adjusted form of the traverse E*abcd*E. It will be noticed that by this method of adjustment the total error is distributed throughout the traverse, so that each line receives only a small linear and angular alteration.

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CHAPTER XIII

PLOTTING TRAVERSE SURVEY BY CO-ORDINATE OR LATITUDE AND DEPARTURE METHOD

THIS chapter deals with the methods of calculating the latitudes and departures of traverse survey lines, the finding therefrom of the co-ordinate distances of the stations referred to two co-ordinate axes, and the plotting of the survey stations by means of these co-ordinates. It deals also with the adjustment of angular errors and closing errors, and consideration is given to the problems involved in altering the bearings of survey lines to suit new co-ordinate axes and in connecting survey lines from one sheet to another. Finally, the significance of closing error is considered and limits of permissible closing error under various conditions are given.

Co-ordinate Method of Plotting Traverse Survey.—The co-ordinate method is the most accurate and satisfactory method of plotting a traverse. It enables the closing error of the field work to be accurately determined before plotting is commenced and to be adjusted if necessary, whereas with the angle and distance methods the closing error is not found till the lines are plotted, and even then it is not known how much of the error is due to the field work and how much has arisen in the plotting. Further, in the co-ordinate method each station point is plotted separately and independently, so that errors due to draughtsmanship are not accumulated. Where reliable results are desired, the labour in working out the co-ordinates by the methods explained in the following pages is far outweighed by the satisfaction which the surveyor has in the proved consistency of his field work and the confidence that none of the work expended in plotting will be wasted.

The co-ordinate method is by far the best to adopt where the work is of such extent as to require to be plotted on separate sheets. When the co-ordinates have been worked out the plotting of the survey can proceed on several sheets at the same time.

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Latitude and Departure.—The term "latitude" denotes the co-ordinate length of a survey line measured parallel to an assumed meridian direction which may be either true north, magnetic north, or any reference direction which may suit the purpose. The term "departure" denotes the co-ordinate length of a survey line measured at right angles to the meridian direction. In Fig. 155 the length AD measured north and south is the latitude of AC, while AB measured east and west is its departure. Similarly the lengths CH and CG are the latitude and departure respectively of the survey line CF.

The magnitudes of the latitude and departure of a survey line are

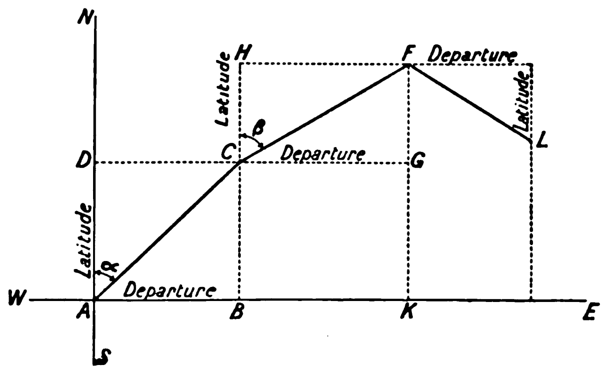

FIG. 155.—Latitudes and Departures of Survey Lines.

found by the principles of trigonometry. If the line AC (Fig. 155) makes the angle α with AN then—

$$\text{Latitude AD} = \text{AC} \cos \alpha$$
$$\text{Departure AB} = \text{AC} \sin \alpha$$

Similarly if line CF makes an angle β with the meridian direction

$$\text{Latitude CH} = \text{CF} \cos \beta$$
$$\text{Departure CG} = \text{CF} \sin \beta$$

Co-ordinates.—The latitudes and departures of the survey lines of a traverse are calculated in order to determine the co-ordinate distances of the stations from two axes. If a north and south line has been adopted as the reference direction for the bearings the co-ordinate axes for *plotting* purposes will in general be taken in the

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north and south and east and west directions. For convenience also one or both of the axes will usually be taken through one of the survey stations. In the figure AN and AE are the co-ordinate axes. The co-ordinates of point C are AB and BC equal respectively to the departure and latitude of the line AC. The co-ordinates of point F are AK and KF. AK is equal to the sum of the departures of the lines AC and CF, while KF is equal to the sum of their latitudes. The co-ordinate of point L along the axis AE will be obtained by adding the departure of the line FL to the co-ordinate AK of point F, while its perpendicular co-ordinate will be obtained by subtracting the latitude of FL from FK. The determination of the co-ordinates of the survey points along one axis is thus accomplished by the successive addition and subtraction of departures,

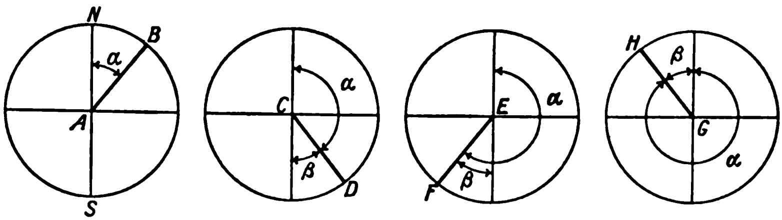

FIG. 156.—Calculation of Latitudes and Departures from Whole Circle Bearings.

while the co-ordinates along the other axis are obtained by the addition and subtraction of the latitudes.

Distinction between North Latitude and South Latitude and between East Departure and West Departure.—In summing up latitudes and departures to obtain the co-ordinates of the stations the surveyor must consider that he is proceeding along the survey lines in succession one way round. Then any line traversed over in a northeasterly direction will be considered to have a north latitude and an east departure. If the direction of motion in passing over a line is towards S.E. the line will have south latitude and east departure. Direction towards S.W. means south latitude and west departure, while N.W. direction means north latitude and west departure. North latitudes and east departures are considered positive and south latitudes and west departures negative. The summations to arrive at the co-ordinates are then made algebraically.

The diagrams in Fig. 156 and the accompanying table illustrate

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the calculation of latitudes and departures from whole circle bearings, and show the proper signs for bearings which occur in each of the four quadrants. The whole circle bearing is represented in each case by a and the reduced bearing required for calculation purposes is represented by β.

CALCULATION OF LATITUDES AND DEPARTURES FROM WHOLE CIRCLE BEARINGS.

Line.	Quadrant.	Reduced Bearing.	Latitude.		Departure.	
			Value.	Sign.	Value.	Sign.
AB	N.E.	$\beta = a$	AB $\cos \beta$	N (+)	AB $\sin \beta$	E (+)
CD	S.E.	$\beta = 180° - a$	CD $\cos \beta$	S (−)	CD $\sin \beta$	E (+)
EF	S.W.	$\beta = a - 180°$	EF $\cos \beta$	S (−)	EF $\sin \beta$	W (−)
GH	N.W.	$\beta = 360° - a$	GH $\cos \beta$	N (+)	GH $\sin \beta$	W (−)

It will be seen that the reduced bearing required for the computations is the angle (less than 90°) which the survey line makes with the meridian direction. For a bearing in the N.E. quadrant no reduction is required. When the whole circle bearing lies between 90° and 180° subtract it from 180° to obtain the reduced bearing. When it lies between 180° and 270° subtract 180° from it, and when it lies between 270° and 360° subtract it from 360°.

Bearings taken with a compass graduated on the quadrant system with zeros at the north and south points require no reduction.

When the angles of the traverse have been calculated by the direct or separate angle method the whole circle bearings of the survey lines must first be calculated in the manner shown on p. 147, and the reduced bearings can then be worked out.

Calculation of Latitudes and Departures.—The method of multiplying the length of the survey line by the natural sine and cosine of the reduced bearing is very laborious and would not be used. In practice the calculations would be made with the aid of either logarithmic tables or traverse tables.

The first requirement, whichever method of calculation is adopted, is to prepare a sketch of the survey lines with stations lettered or numbered and with all bearings and lengths plainly figured along the lines to which they refer. The sketch will be all the more useful

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if it is roughly plotted to scale with a protractor. It will then show the true form and disposition of the lines and will be useful in arranging the proper position of the work for plotting. It can also be used to check roughly the calculated latitudes and departures, and by showing at a glance whether a given line has north or south latitude or east or west departure it enables mistakes in sign to be avoided.

The traverse shown on p. 160 has been taken as an example to illustrate the calculation of latitudes and departures by the use of logarithmic tables and a convenient method of setting down the figures and making the calculations is shown in the following table.

CALCULATION OF LATITUDES AND DEPARTURES.

Length and Bearing.	Reduced Bearing.	Log Length and Log Sine.	Departure.	Log Length and Log Cos.	Latitude.
AB = 575 90° 0′	90° 0′ E.	—	575 E.	—	0
BC = 660 46° 0′	N. 46° 0′ E.	2·81954 9·85693 2·67647	474·7 E.	2·81954 9·84177 2·66131	458·5 N.
CD = 515 266° 0′	S. 86° 0′ W.	2·71181 9·99894 2·71075	513·7 W.	2·71181 8·84358 1·55539	35·9 S.
DE = 393 309° 0′	N. 51° 0′ W.	2·59439 9·89050 2·48489	305·4 W.	2·59439 9·79887 2·39326	247·3 N.
EA = 707 199° 0′	S. 19° 0′ W.	2·84942 9·51264 2·36206	230·2 W.	2·84942 9·97567 2·82509	668·5 S.

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In the first column the measured values of the length and bearing of each survey line are entered. The reduced bearings referred to the north and south meridian are then worked out and entered in the second column. The third column contains for each survey line the log of its length and the log sine of its reduced bearing and their sum which is the log of the departure. In the fourth column is entered the departure corresponding to the logarithm found in the third column. The last two columns contain the figures for the calculation of the latitudes.

It will be noticed that the bearing of AB is 90° or its direction is due east. This line, therefore, requires no calculation as its departure is just equal to its length and it has no latitude.

It is essential to rapid and accurate work that the operations should be carried out systematically in groups, each group being completed for all of the lines before the next is commenced. The proper order of operations for the tabulated method of calculation shown above is as follows: The reduced bearings are worked out for all the lines and the letters designating the quadrants in which they occur are written down. The logarithms of all the lengths are then looked out from the tables and the value for each survey line is inserted both in the log sine column and in the log cosine column. Log sines and log cosines of the reduced bearings are then abstracted from the tables and inserted under the logs of the lengths. Next, all the additions of the logarithms are made, thus obtaining the logarithms of the latitudes and departures, and from the tables the numbers which correspond to these logarithms are found, thus obtaining the latitudes and departures of the lines.

All the figures and calculations of the table should be carefully checked before any use is made of the latitudes and departures, as it need hardly be emphasised that a mistake in the figures renders the results worse than useless.

Traverse tables give the latitudes and departures of survey lines for angles between 0° and 90°, and lengths of 1, 2, 3, up to ten units. The values given are simply the natural sines and cosines of the angles multiplied by the figures 1, 2, 3, &c. To be of use for ordinary surveying purposes they must be given for each minute of angle and to the fourth significant figure. To find by means of the tables the latitude of a line, say, 473 ft. long write down the

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tabular values for lengths of 400, 70 and 3 ft. and add them together.

The following is a portion of a traverse table for the angle 24° 15′ :—

	1 Lat.	1 Dep.	2 Lat.	2 Dep.	3 Lat.	3 Dep.	4 Lat.	4 Dep.
24° 15′ ...	0·912	0·411	1·824	0·821	2·735	1·232	3·647	1·643

The latitude and departure of a line having a length of 420·3 ft. and a reduced bearing of 24° 15′ would be found as follows, making use of the above line of the tables :—

	Latitude.	Departure.
400 ..	364·7 ..	164·3
20 ..	18·24 ..	8·21
0·3 ..	0·27 ..	0·12
420·3	383·2	172·6

For illustration purposes the separate lengths for which the values are abstracted from the tables have been shown in the first column above. In the actual working out of the latitudes and departures these lengths are not written down.

If a survey line has been measured to the nearest tenth of a foot it serves no useful purpose to go beyond the first decimal place in working out the latitudes and departures. Thus in the above example the latitude and departure for a length of 0·3 ft. as got from the table are 0·2735 and 0·1232 respectively, but it is only necessary to write down 0·27 and 0·12 in order to obtain the desired result.

Calculation of Co-ordinates.—In order that the co-ordinates of the stations may be calculated, one of the stations must be chosen as a starting point and its position fixed with reference to the co-ordinate axes. That is, suitable values must be fixed for the co-ordinates of one of the stations. In the example for which the latitudes and departures have been worked out station A has been taken as the starting point, its co-ordinates being made 0, 0. The calculations for determining the co-ordinates of the other stations are shown in the following table :—

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CALCULATION OF CO-ORDINATES.

Line.	Departure.		Latitude.		Corrected Departure.		Corrected Latitude.		Station.	Co-ordinates.	
	E.	W.	N.	S.	E.	W.	N.	S.		N.	E.
									A	0·0	0·0
AB	575	—	—	—	574·9	—	—	—	B	0·0	574·9
BC	474·7	—	458·5	—	474·6	—	458·1	—	C	458·1	1049·5
CD	—	513·7	—	35·9	—	513·8	—	36·0	D	422·1	535·7
DE	—	305·4	247·3	—	—	305·5	247·0	—	E	669·1	230·2
EA	—	230·2	—	668·5	—	230·2	—	669·1	A	0·0	0·0
	1049·7	1049·3	705·8	704·4	1049·5	1049·5	705·1	705·1			
	Error 0·4		Error 1·4								

In the above table the east and west departures and north and south latitudes as calculated are first entered in the separate columns shown. Each column is then summed up. If the lengths and angles have been measured with perfect accuracy and no mistake has been made in the figures the sum of the east departures should, for a closed traverse, be equal to the sum of the west departures, and likewise the sums of the north and south latitudes should balance. In general it will be found that the sum of the east departures is not quite equal to the sum of the west departures and that there is also a difference between the sums of the latitudes. If the difference between the totals is so small as to be inappreciable in the length of a plotted survey line the co-ordinates may be worked out directly from the figures entered in the first four columns. Where, however, the difference is large enough to have a definite size if plotted to the scale of the plan the values of the latitudes and departures must be adjusted before the co-ordinates are calculated. To make clear the distinction between appreciable and inappreciable difference suppose that the scale of the plan is 1 in. = 100 ft. A surveyed length of 1 ft. will be represented by $\frac{1}{100}$ in. on the paper, and this is about the smallest amount which can be definitely plotted. A difference, therefore, of 0·3 ft. in the sums of the latitudes or departures might be ignored, but a difference of 2·3 ft. would be quite appreciable on the plan, and such an error would

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require to be distributed throughout the survey lines before the co-ordinates were calculated. In the example tabulated above the east departures exceed the west departures by 0·4 ft. and the north latitudes exceed the south latitudes by 1·4 ft. In correcting the latitudes and departures it has been assumed that the error will be equally distributed throughout the lines. Half the total error in departures or 0·2 ft. will, therefore, fall to be distributed amongst the east departures and the other half amongst the west departures. The correction in this case has been made by subtracting 0·1 ft. from the east departures of each of the lines AB and BC and by adding 0·1 ft. to the west departures of lines CD and DE. Similarly the sum of the north latitudes requires to be reduced by 0·7 ft., and the sum of the south latitudes requires to be increased by an equal amount. The actual amount of the correction applied to each line should be made proportional to its latitude.

The corrected latitudes and departures are entered in the four additional columns as shown. In practice these additional columns are usually dispensed with and the corrected latitudes and departures are written in red ink above the figures in the latitude and departure columns. The co-ordinates of point A are 0, 0. The co-ordinates of the other points B, C, D, &c., in the east and west direction are arrived at by algebraic summation of the corrected departures of the lines AB, BC, CD, &c. The co-ordinates in the north and south direction are found in a similar manner.

In this example the axes have been so chosen that the whole of the survey lines lie in the north-east quadrant with respect to the origin. This makes the co-ordinates of all the points positive. For the sake of simplicity it is in general desirable to arrange that the survey points shall all occur in the same quadrant, but not necessarily any particular one.

For purposes of illustration the co-ordinates have been worked out in the table on p. 175 for the case in which both axes pass through the polygon. Station D has been taken as the starting point, its co-ordinates being fixed at E. 200, N. 300.

The lines and points must as before be taken in consecutive order round the polygon from the starting point. Care is required in making the subtractions where the departures change from E. to W. and where the latitudes change from N. to S. The arithmetical work is checked by noting if the co-ordinates of the

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Line.	Corrected Departure.		Corrected Latitude.		Station.	Co-ordinates.	
	E.	W.	N.	S.		E. & W.	N. & S.
					D	200·0 E.	300·0 N.
DE	—	305·5	247·0	—	E	105·5 W.	547·0 N.
EA	—	230·2	—	669·1	A	335·7 W.	122·1 S.
AB	574·9	—	—	—	B	239·2 E.	122·1 S.
BC	474·6	—	458·1	—	C	713·8 E.	336·0 N.
CD	—	513·8	—	36·0	D	200·0 E.	300·0 N.

starting point found at the conclusion after working right round the polygon agree with the values taken at starting.

Adjustment of Angular Errors.—It has been assumed in working out the preceding example that there is no error in the angular work. In observing angles in the ordinary manner with a theodolite reading to single minutes an error of some fraction of a minute may be expected to occur at each angle. In a series of angles of a traverse these small errors will not all tend in the same direction so that the total error will not accumulate in direct proportion to the number of the sides, but it may nevertheless soon become appreciable. Thus in a polygon of a dozen sides in which all the interior angles have been read by the direct or separate method it need not be expected that the sum of the angles will amount exactly to the proper number of right angles. An error of three minutes would not indicate careless work and would not necessitate the angles being read over again. Before proceeding to calculate latitudes and departures, however, the angles would require to be adjusted so as to add up correctly, by distributing the error throughout the work.

Where whole circle bearings of the traverse lines have been read in the field it will generally be found that a small error has crept in round the polygon. If the error is within permissible limits the bearings should be adjusted so as to make the final bearing of the first line agree with its original bearing. The method of making the adjustment is illustrated in the following table for a set of imaginary bearings which show a difference of three minutes between the first and final bearings of the commencing line AB.

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ADJUSTMENT OF ANGULAR ERRORS.

Line.	Forward Bearing.	Correction.	Adjusted Bearing.
AB	90° 10′	—	90° 10′
BC	46° 13′	—	46° 13′
CD	253° 15′	− 0° 1′	253° 14′
DE	259° 47′	− 0° 1′	259° 46′
EF	278° 12′	− 0° 1′	278° 11′
FG	221° 35′	− 0° 2′	221° 33′
GH	182° 16′	− 0° 2′	182° 14′
HJ	155° 49′	− 0° 3′	155° 46′
JK	94° 53′	− 0° 3′	94° 50′
KA	143° 49′	− 0° 3′	143° 46′
Check AB	90° 13′	—	90° 10′

As the alteration of the bearing of any one line affects the bearings of all lines subsequently observed, so the effect of the correction of a bearing requires to be carried through all the following bearings. The adjustment is, therefore, not quite the same as when the separate interior angles are read, as in the latter case only three angles would be altered. In adjusting the bearings the assumption is made that the error of three minutes has accumulated uniformly in working round from side AB to side KA, and the correction is made accordingly, but without going to fractions of a minute.

If a very exact adjustment is required, as might be the case where angles have been read to fractions of a minute with an accurate theodolite, the correction will not be constant over several lines, but will be different for each side. If E is the total angular error and N is the number of corrections required (one less than the number of sides) then taking the sides in order round the polygon, but omitting the first side which remains unaltered, the corrections will have the following successive values: $\frac{E}{N}$, $\frac{2E}{N}$, $\frac{3E}{N}$, &c. These values may be worked out to seconds of angle, but the correction need not be made with any greater degree of precision than has been adopted in observing the angles.

Adjustment of Latitudes and Departures.—The method of adjusting latitudes and departures which has been applied in the example shown on p. 173 is simple and quite suitable for ordinary purposes.

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On the assumption, however, of equal chance of error in the angular and linear work the proper method of adjustment would be to distribute the total error of latitude throughout the lines, making each correction proportional to the length of the survey line to which it applies, and similarly with the error of departure. The corrections are then given by the following equations:—

$$\frac{\text{Correction in latitude of a side}}{\text{Length of side}} = \frac{\text{Total error in latitude}}{\text{Total length of traverse}}.$$

$$\frac{\text{Correction in departure of a side}}{\text{Length of side}} = \frac{\text{Total error in departure}}{\text{Total length of traverse}}.$$

Weighting the Survey.—Where it is known that certain portions of the work have been measured with greater accuracy than others the corrections should be judiciously placed at the points where the probability of error is greatest. For example, in surveying large areas the traverse polygons are commonly arranged with one side extending between two stations of a triangulation system. The length of this side will have been determined very precisely, and hence in adjusting the polygon the corrections should be confined to the other sides.

In a traverse where some lines run over level ground and others over steep or rough ground, the probability of error is much greater in the latter than in the former, and this should be taken into account in adjusting the latitudes and departures. It should also be borne in mind that on rough or steep ground the tendency is always to measure the lines too long, and hence any adjustment made on such lines should preferably be in the direction of making them shorter.

Graphical Method of Correcting for Errors in Chainage only.—In many cases, especially in surveys over uneven country, the angular work will be much more accurate than the linear work, and in such cases it would seem desirable to confine the adjustment to the linear work alone, the traverse being made to close by a simple alteration of the lengths of the lines. The method is rather laborious. It is necessary in the first place to have a fairly accurate diagram of the survey lines (such as may be plotted with a protractor), and to have the total errors in latitude and departure and the total magnitude of

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the closing error worked out. The latter is equal to $\sqrt{E_l^2 + E_d^2}$. where E_l and E_d represent the total errors in latitude and departure respectively. The closing error is the same, no matter which point of the polygon is taken as the starting point. Fig. 157 shows how the closing error might occur in a five-sided polygon, according as

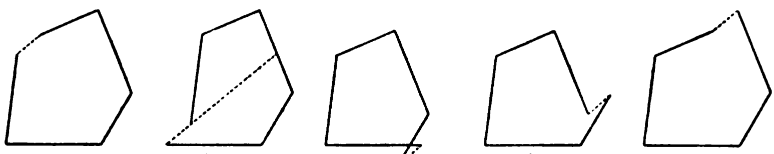

FIG. 157.—Closing Error of a Polygon.

each angle was in turn made the starting point for the plotting. It is evident that for any given polygon there will be at least one angle at which the direction of the closing error produced will cut the polygon into two roughly equal halves, and in many cases there will be two such angles situated on opposite sides of the polygon. In Diagram (*b*) (Fig. 157) the direction of the closing error passes roughly through the middle of the polygon. In making the linear adjustment of the traverse lines it is necessary to find the angle at which this condition is fulfilled.

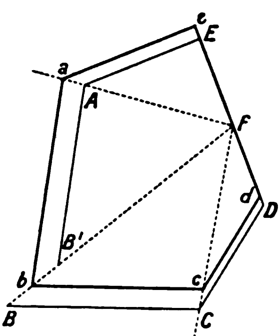

FIG. 158.—Linear Adjustment of Closing Error.

Fig. 158 illustrates the method of making the linear correction of the polygon, the closing error being BB′ (shown very much exaggerated in the figure) and its direction B*f* dividing the polygon into two portions, forming the separate polygons BCD*f* and B′AE*f*. The error BB′ is bisected in point *b*. The polygon B′AE*f* is enlarged to form the similar figure *baef*, the sides *ba* and *ac* being parallel to B′A and AE respectively. The length of each side is increased by the fraction $\frac{bB'}{B'f}$ of its original length. In the same way the polygon BCD*f* is reduced to form the similar figure *bcdf*, each side being diminished by the fraction $\frac{Bb}{Bf}$ of its original length. The fractional reduction in the length of the sides of the polygon BCD*f* is for all

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practical purposes the same as the fractional increase in the length of the sides of the polygon B′AE*f*. *bcdfea* is the complete corrected polygon. The bearings of its sides have not been altered, the whole adjustment having been effected by slight alteration of the length of the sides.

Fig. 159 illustrates the method of finding by calculation from the co-ordinates and by graphical construction the direction and magnitude of the closing error and the proper station of the traverse at which to deal with it. BCDEA represents the polygon plotted with the protractor to a convenient scale. The final side of the polygon is simply joined up between the first and last plotted points, the closing error being meantime ignored. OX and OY represent the directions of the co-ordinate axes. Plot to a much enlarged scale along OX the distance O*g* equal to the total calculated error in departure. O*g* will be plotted to the right or left of the origin, according as east or west departures are the greater. Similarly lay off *gh* to represent the total calculated error in latitude. Then O*h* represents the total closing error in magnitude and direction. Draw a parallel to O*h* through that station which will cause the polygon to be most nearly bisected into equal halves, station B in the figure. Scale off the length of B*f*. Then, for the case shown, the lines BA, AE and E*f* require to be each increased by the fraction $\frac{\text{half total closing error}}{Bf}$ of their measured length and the lines BC, CD, and D*f* require each to be diminished by the same fraction of their original length.

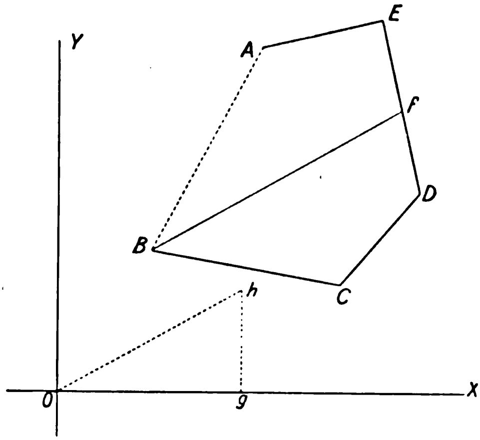

FIG. 159.—Linear Adjustment of Closing Error.

The alteration of the length of any survey line will cause a proportional alteration in its latitude and departure, so that instead

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of dealing with the actual lengths of the lines their latitudes and departures, which have been already calculated, may be altered in the above proportion. Then, if the operations have been correctly performed, the sum of the north latitudes should be equal to the sum of the south latitudes, and likewise the sums of the east and west departures should balance.

In the line ED the part E*f* requires to be increased and the part *f*D diminished. If point *f* is near the middle of ED there will, therefore, be no alteration necessary in the total length of ED or

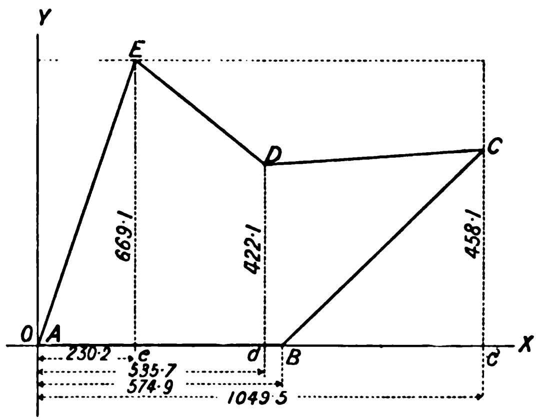

FIG. 160.—Plotting Survey Stations by Co-ordinates.

in its latitude and departure. In general, the total correction of the line ED will be equal to

$$(\mathrm{D}f - f\mathrm{E})\ \frac{\text{half total closing error}}{\mathrm{B}f}$$

Plotting Survey Stations by Co-ordinates.—The plotting of the positions of the survey stations after the co-ordinates have been calculated is very simple. Fig. 160 illustrates the plotting of the polygon whose co-ordinates have been already worked out on p. 173. The easterly co-ordinates, as given in the last column of the table, are marked off to scale along the axis OX from the origin O. O*e* is the easterly co-ordinate of point E, O*d* of point D, and so on. Perpendiculars are erected at the points *e*, *d*, &c., with lengths equal to the co-ordinates of the stations, as given in the second last column of the table. *e*E is equal to the north co-ordinate of point E,

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*d*D is equal to the north co-ordinate of point D, and so on. The points E, D, &c., so plotted are then joined up by fine pencil lines, which form the sides of the traverse. The accuracy of the plotting is checked by scaling the lengths of these sides and noting whether there is agreement with the measured lengths of the survey lines. If no adjustment has been necessary in the latitudes and departures the agreement should be almost perfect. If some adjustment has been required there will necessarily be a slight discrepancy, which, however, should be insignificant.

The most fruitful source of inaccuracy in plotting by co-ordinates lies in the construction of the perpendiculars. Straight-edges and set-squares should not be used until they have been tested and proved to be true. Long perpendiculars should not be constructed by drawing a portion of the length with a set-square and then extending this line. The necessity for plotting a large number of long perpendiculars can be avoided by special devices, the best method where the stations are numerous being to lay out a bounding rectangle with its sides accurately at right angles to each other and to sub-divide this into smaller squares or rectangles, with sides having a convenient round length. Each station will be plotted by small perpendiculars constructed with the set-square from the sides of the rectangle in which it occurs.

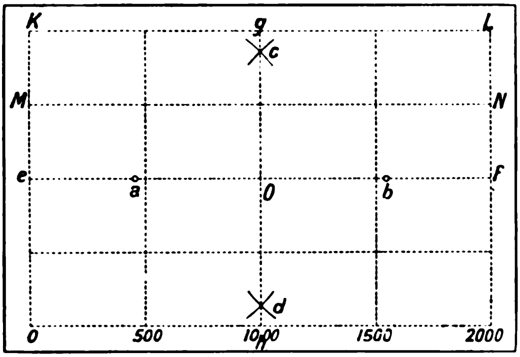

FIG. 161.—Construction of Bounding Rectangle.

A good method of constructing the bounding rectangle and subdividing it into squares is illustrated in Fig. 161. The base line *ef* is drawn along the middle of the paper. Perpendiculars are erected with the set-square near each end of this line, and along these the lengths of the sides of the small squares are marked off to scale. Lines MN, KL, &c., drawn through these points will be parallel to the line *ef* and at the correct distance apart (for all practical

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purposes), even although the perpendiculars drawn at the ends of the line *ef* were not absolutely square. To construct an accurate perpendicular to the line *ef*, two points, *a* and *b*, are taken at equal distances on each side of the centre of the paper. The distance *ab* should be rather less than the width of the rectangle *gh*. Using the beam compasses or a strip of stout paper and taking a suitable radius, strike from the centres *a* and *b* intersecting arcs to give the points *c* and *d* near the top and bottom of the sheet. The length of the radius should be such as to make the arcs cross each other at about 90°. The line *gh* drawn through the points *c* and *d* will be an accurate perpendicular to the line *ef*. From *g* and *h* scale off in both directions lengths equal to the sides of the small squares, which should not exceed 10 ins., and draw vertical lines through corresponding points on the top and bottom sides. These lines will also be perpendicular to the line *ef* and to the other horizontal lines. The accuracy of any square may be tested by measuring the lengths of both the diagonals and noting whether they are the same. Wherever it can be done without detriment to the plan the sides of the squares should be inked in with very fine lines. The squares will shrink and expand with the paper, and will form a valuable permanent scale over the whole area of the plan.

If the plan is in the form of a long roll a straight base line should be set out along the middle of its length by means of a stretched thread. The sides of the squares will be scaled off along this line and through points chosen at intervals perpendiculars will be constructed as described above, their distance apart being dependent on the length of straight edge available. The sub-division into squares will then be carried out in the manner described above.

Alteration of Bearings to suit New Co-ordinate Axes.—It may happen that it is not convenient to make one of the co-ordinate axes coincide with the direction of the reference meridian. The case is illustrated in Fig. 162, where the arrow head represents the direction of the meridian to which bearings have been referred, and it is desired to plot the area shown on a piece of paper represented by the dotted rectangle OXZY. It is evident that for plotting purposes the axes should be taken parallel to the directions OX and OY. Let the direction of OY be taken as a new meridian. Its bearing must be altered to 0° 0′, and the bearings of all the

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survey lines must be altered correspondingly. If the bearing of OY with respect to the former meridian is α°, then the angle α must be subtracted from the whole circle bearings of all the lines to make them refer to the new meridian OY. If the bearing of any line is less than the angle α add 360° to it before making the subtraction. Latitudes and departures worked out from the new bearings will be parallel to the lines OX and OY respectively, and the co-ordinates can be worked out with respect to these or any parallel lines as axes.

Connecting Survey Lines from one Sheet to another.—The survey of a large area may be readily plotted by the co-ordinate method on separate sheets or rolls instead of on one large sheet. The work on each sheet is confined within rectangular boundary lines which form common junctions between adjacent sheets. Where the ends of a survey line occur on different sheets a special device or calculation is required to enable the two portions of the line to be plotted. The simplest method is to draw on a piece of tracing paper a single line, corresponding to the common junction line, and to trace a portion of the reference squares on either side of this line. Apply the tracing to one of the sheets, making the junction line and sides of the squares coincide and transfer the position of the station to the tracing. Similarly, apply the tracing in a corresponding position on the other sheet and mark the position of the second station. Draw a line connecting the two stations on the tracing paper. Scale off its length and note whether it agrees with the measured length. If the result is satisfactory apply the tracing again in succession to the two sheets and prick through the points where the survey line crosses the junction line. The two portions of the survey line will be got by joining up these points to the survey stations. A further check may now be applied by measuring the

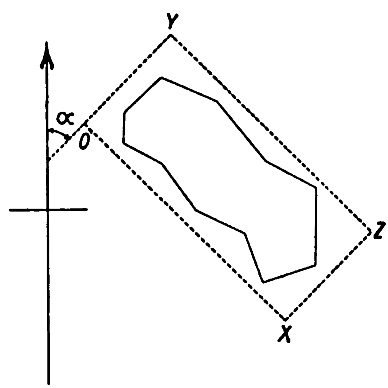

Fig. 162.—Alteration of Bearings.

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separate lengths of these portions and seeing whether they sum correctly. The method of finding by calculation the point where a survey line crosses the boundary line is explained under "Traverse Problems" on p. 214.

Closing Error and Limits of Error.—The amount of the closing error depends in great measure on the *consistency* with which the field work has been carried out. The magnitude of the closing error is no criterion of the absolute accuracy of a traverse. For example, a polygon may have been measured with a chain of incorrect length. If the length of the chain has remained constant throughout, its accuracy will not have any effect in causing closing error, but the resulting survey will nevertheless be inaccurate. Also cases will often occur where the effect of sloping ground may be neglected in chaining the survey lines without giving rise to closing error as any errors introduced in chaining outwards may be balanced by corresponding errors in returning. If, however, constant errors, such as result from incorrect length of chain, have been as far as possible eliminated and consistent accuracy has been aimed at throughout, the closing error will to some extent be an indication of the accuracy of the work.

Using an ordinary good theodolite reading to single minutes the error in any single observation should be less than half a minute. An error of thirty seconds in the bearing of a line will cause a lateral displacement of the extreme station to the extent of about $\frac{1}{6800}$ of the length of the line. This amount of displacement is small compared with that which is unavoidable in ordinary chaining. On undulating ground using the ordinary jointed chain an error of $\frac{1}{1000}$ may represent quite good work while with the steel band the error should not exceed $\frac{1}{2000}$. On level ground and with special precautions much smaller errors than these may be obtained in chaining, but generally the angular work will be relatively more accurate than the linear. With the assumption of perfect accuracy in the bearings the closing error expressed as a fraction of the perimeter of the polygon should never be as great as the linear error and should hardly exceed half its amount. Thus, if the probable error in chaining is $\frac{1}{1000}$ the closing error (due to chaining) should not exceed $\frac{1}{2000}$. On undulating ground a total closing error of $\frac{1}{1000}$ when the chain is used and of $\frac{1}{2000}$ when the steel band is employed will

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represent fairly good work. Closing errors should not exceed half these amounts on level ground. Special precautions in the measurement of lengths and angles are necessary if a closing error of less than $\frac{1}{5000}$ is to be consistently attained.

The limit of error to be set for any particular survey should be made to depend on the purpose for which the survey is required and on the time and money available for the work. The limit of accuracy attainable on paper should also be kept in mind. It is very easy to measure a survey line with a steel band with an error of less than $\frac{1}{4000}$. It is quite impossible to measure by scale a distance on a plan to such a degree of accuracy owing to unequal contraction and expansion of the paper.

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CHAPTER XIV

TRIANGULATION

IN this chapter the principles of triangulation by which, starting from a measured base line, the positions of a number of points are fixed from measurements of angles alone, are considered and the application of those principles on a modest scale in practice is also dealt with. The important sources of error in reading angles are also considered.

Triangulation.—Starting from the base line AB (Fig. 163), whose length has been accurately measured, the position of a point C may be fixed by observing the angles at A and B of the triangle ACB. The lengths of the sides AC and CB can then be calculated by the principles of trigonometry and these sides may, therefore, be used as new bases from which to fix other stations such as D and E by angular measurements in the same way. The sides of the triangles ADC and BEC may in turn be used as base lines from which to fix additional points, and proceeding in this way a connected system of triangles may be built up the positions of whose corner points are determined by angular measurements alone. This method of fixing the positions of a series of points over an area to be surveyed is known as triangulation.

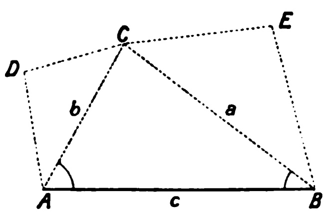

FIG. 163.—Fundamental Problem in Triangulation.

Purpose of Triangulation.—By the triangulation method the fixing of an additional station whether it be half a mile or twenty miles distant is effected by the observation of a few angles. The accuracy of the angular work is practically uninfluenced by the distance or the irregularity of the intervening ground, being determined almost solely by the precision of the instrument employed and the care and precautions bestowed on its use. It need hardly

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be pointed out that the results attainable by triangulation cannot be equalled or approached in practice by any method involving linear measurements owing to the amount of labour necessary in chaining long distances and the unavoidable inaccuracy where the ground is uneven. But while triangulation is thus eminently suitable for the fixing of a series of points distributed at considerable intervals over a large area it is not adapted to the location of the numerous minor stations at close intervals required in the picking up of detail. For this purpose some other method such as traverse surveying will usually be found suitable. The methods of surveying adopted for the location of detail are, however, not in general sufficiently reliable when applied to large areas, and hence the purpose of triangulation becomes evident. That purpose is to locate accurately over the area to be surveyed a series of check points whose positions so determined are accepted as correct. The minor survey lines for the location of the details are then connected as directly as possible to these points and any closing error found in proceeding from one check point to another is corrected by adjusting the intermediate lines and not by any alteration of the positions of the triangulation stations. The error in the detail surveying is thus prevented from accumulating, being confined to the small amount which can occur within any single triangle and this amount can easily be kept within reasonable limits.

For the survey of a *country* a network of main triangles is set out having sides of twenty miles or more in length. The main triangles are then divided and sub-divided into smaller triangles the ultimate minor triangulation to which the detail surveying is connected having sides of an average length of usually less than one mile.

In the main triangulation instruments of great precision are employed, extraordinary precautions are necessary in observing the angles and in adjusting their values, and the spherical form of the earth must be taken into account. For the minor triangulation instruments of ordinary size (5 ins. or 6 ins.) suffice, but greater precautions are taken in the reading of angles and the adjustment of their values than would be taken in the case of a simple traverse survey. The effect of curvature of the earth is inappreciable in minor triangulation and may be entirely neglected.

In the following pages consideration will only be given to the subject of minor triangulation on a small scale such as would be

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suitable for a survey extending over a few square miles, appropriate lengths for the sides of the triangles being 2,000 to 5,000 ft.

Precautions to Ensure Accuracy.—As regards the shape of the triangles the equilateral form will give the best results, and the practical rule is to have no angle of a triangle less than 30°, which necessarily also means that no angle can be greater than 120°.

The length of one side of a triangle being known the observation of the two adjacent angles will enable the lengths of the remaining sides to be calculated. It is not necessary to read the third angle of the triangle as its value can be found by deducting the sum of the other two from 180°, but the reliability and accuracy of the work will be enhanced by reading the three angles separately. If their sum is found to differ by a slight amount from 180° a correction equal to one-third of the difference should be added to or subtracted from each angle so as to obtain adjusted values which will add up exactly to 180°.

Many of the stations in a triangulation, such as point C (Fig. 164) will form the common apex of several surrounding angles. The sum of the angles around any such station is equal to 360°, and where any slight discrepancy is found in summing up the observed values a suitable correction must be applied.

Calculation of Lengths of Sides of a Triangle.—The calculation of the lengths of the sides is accomplished by the aid of simple trigonometry. In any triangle, such as ABC (Fig. 163), the following relationship exists between the lengths of the sides and the sines of the angles :—

$$\frac{a}{\sin A} = \frac{b}{\sin B} = \frac{c}{\sin C}$$

so that

$$a = \frac{c \sin A}{\sin C} \text{ and } b = \frac{c \sin B}{\sin C}$$

If, therefore, the angles A, B and C, and the length of side c are known, the lengths of sides a and b can be calculated from the above formulas. Logarithms must be used in making the calculations in order to avoid laborious multiplication and division. The formulas then become—

$$\text{Log } a = \log c + \log \sin A - \log \sin C$$
$$\text{Log } b = \log c + \log \sin B - \log \sin C$$

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It will be noticed that the terms log c — log sin C are common to both equations and attention should be paid to this in arranging the figures so as to avoid unnecessary calculation. As an example, the lengths of the sides a and b will be worked out for a triangle having a known side $c = 4287{\cdot}3$ ft. long and angles as shown in the first column of the following table :—

The figures are set down in tabular form as follows :—

CALCULATION OF SIDES OF TRIANGLES.

Particulars.	Logs and Log Sines.		Logs of Sides.	Sides.
c = 4287·3 ft.	3·63218			
A = 61° 21′ 10″	—	9·94329	3·58028	3804·4 ft. = BC
C = 81° 29′ 30″	9·99519	3·63699		
B = 37° 09′ 20″	—	9·78102	3·41801	2618·2 ft. = AC

In the second column the log of the known side and the log sine of the angle opposite to it are inserted. In the third column the log sines of the two remaining angles are written down each in its proper line. The lower number in the second column (log sin C) is then subtracted from the upper number (log c) and the result, 3·63699, which is equal to log c — log sin C, is written down between the numbers in the third column. The two upper numbers in the third column are then added together and the result, which is equal to log sin A + log c — log sin C and, therefore, equal to log a, is placed in the fourth column. The addition of the two lower numbers in column 3 likewise gives the log of side b, and this is also placed in the fourth column. The lengths of the sides are obtained by looking up from the tables the numbers corresponding to the logs of the sides.

By arranging the work according to the above plan the lengths of the sides are arrived at with the least possible amount of calculation and figuring.

For a survey extending over a few square miles it will be quite sufficient to employ five-figure logarithms as in the above example. For much larger areas logarithms to six or seven places of decimals might be used according to the magnitude of the triangulation and the degree of accuracy aimed at.

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Closing Error in Triangulation.—Even when the angles of each triangle have been adjusted to sum up to 180° and the angles round each station have been corrected so as to sum to 360° there will remain room for a certain amount of error in each angle. In a minor triangulation the amount of error in an angle may be only a few seconds and should not in the most extreme case reach half a minute. In consequence of these slight angular errors the calculated length of a side at some distance from the base line will vary slightly according to the route which has been taken in working round to the line. Thus, referring to Fig. 164, the sides of the triangles 1, 2, 3 may be successively calculated starting from the base line AB and a value for the length of the side CF thereby obtained, thus fixing a position for station F. But by calculating the sides of triangles 4 and 5 a second value will be got for the length of side CF. This value will usually differ slightly from that previously obtained, with the result that the position of the station as fixed from triangle 5 may be at F′ instead of at F. The distance FF′ is analogous to the closing error in a traverse polygon.

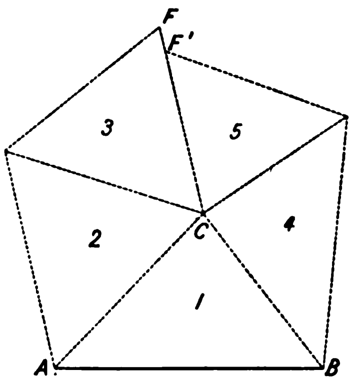

FIG. 164.—Closing Error in Triangulation.

It is possible to adjust the angles in a set of triangles which form a polygon about a central point so that there will be no closing error, but in a small triangulation it will not generally be necessary to do this. It will be sufficient to take the length of the side as equal to that value which has been obtained by the most direct route.

Field Work.—The principal field operations for a triangulation consist of :—

(*a*) The establishing and measuring of the base line.
(*b*) The fixing of the positions of the stations.
(*c*) The reading of the angles with the theodolite.

Measuring the Base Line.—The base line must be carefully and accurately measured, as any error in its length will affect the

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calculated lengths of all the other sides. The degree of accuracy to be aimed at will depend on the purpose and magnitude of the triangulation and should also be made consistent with the accuracy attainable in the angular work.

In the following description of the measurement of a base line the precautions to ensure accuracy are such as have been used for a small triangulation where the purpose was simply to obtain an accurate plan.

Smooth level ground is desirable for the location of the base line, but a uniform slope is not very objectionable. The length of the base line should preferably be about equal to the average length of the sides, but a shorter length must of necessity often suffice. The base line may be located at any place within the area to be surveyed where the conditions are favourable for its accurate measurement. It is a good plan to have the base line near one end of the survey and to measure also a check base line forming a side of a triangle near the other end.

The measurement will be made with a steel band or steel tape. Sufficiently accurate results cannot be obtained with the chain. The steel band or tape should before use be carefully compared with a reliable standard and its error noted and also the temperature at the time of the observation.

The base line will be ranged out with the theodolite and marked at intervals close enough to prevent appreciable deviation from the true course in chaining. The ends of the line may be marked by a stout wooden peg driven into the ground and having a nail in the head to indicate the exact point. If the line is on a slope the vertical angle of inclination should be read with the theodolite by sighting to a mark at the extreme station fixed at the height of the telescope.

The chaining will proceed directly along the surface of the ground. On soft ground the chain lengths will be marked by sticking in arrows and on hard ground, such as a road or footpath, by making a fine scratch on the surface and leaving the arrow beside it. Considerable care must be exercised in planting the arrows and in holding the handle up to them, as it is easy to omit the thickness of an arrow at each chain length. The band must be kept in good line and straight between its ends. A steady and constant pull of 12 or 15 lbs. should be applied to it at the instant of fixing the arrow.

The average temperature during the chaining should be observed,

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and if it is found to be much different from that at which the band was tested a correction of the measured length should be made.

The length of the base line should be chained at least twice, once in each direction. If a considerable discrepancy, say, more than 1 in 10,000, were found to exist between the two determinations the line should be rechained again in each direction. The result might be to show that one of the first determinations was obviously inaccurate. The mean of the others would then be accepted as the most probably accurate result.

The mean length determined in this manner requires to be corrected for (*a*) error in length of band, (*b*) temperature, and (*c*) slope of ground.

To illustrate the method of making the corrections, let us take the following example: The mean length of a base line as chained directly along the surface of the ground was 4368·7 ft. The slope of the ground was 1° 14′. The band, when compared with a standard at a temperature of 50° F., was found to be 100·03 ft. long. The base line was chained at a temperature of 62° F. It is required to find the true length of the base line.

Each length which has been booked in the result as 100 ft. is in reality 100·03 ft. The error due to incorrect length of chain will, therefore, be eliminated if the result is multiplied by 1·0003.

The difference of temperature causes an alteration in the length of the band and the correction must, therefore, be applied as for incorrect length. A change of 1° F. causes an alteration in the length of a piece of steel of about 0·000007 of its length. An increase of 12° in temperature will, therefore, cause an expansion of the band to the extent of 0·000084 of its previous length, or the altered length will be 1·000084 of the original length. The total error of the band at time of chaining will, therefore, be 0·0003 + 0·000084, or 0·000384 of its length. The true length of the line measured along the slope will, therefore, be 4368·7 ft. × 1·000384.

If a length L be measured along a slope of A°, the true horizontal distance X will be equal to L cos A.

$$\text{Cos } 1^\circ\ 14' = 0{\cdot}99977.$$

Therefore the true horizontal length of the base line is 4368·7 ft. × 1·000384 × 0·99977 = 4369·4 ft.

Broken Base Line.—It may happen that a long enough stretch of

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suitable ground cannot be found for the accurate measurement of a base line in a single straight length. This difficulty may sometimes be overcome by arranging the base line in two portions, making an angle with each other, as in Fig. 165. The actual base is then the line AC and its length must be calculated. The separate portions AB and BC would be chained and the true length of each determined in the manner above described. When the theodolite was set up at the points A and C for the purpose of ranging out the lines, the opportunity would be taken to read the angles BAC and ACB. If BD is perpendicular to AC, then AD = AB cos A and DC = BC cos C, so that the whole length AC is equal to AB cos A + BC cos C.

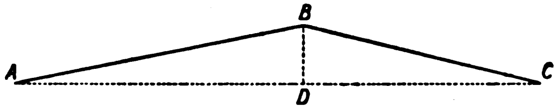

FIG. 165.—Broken Base Line.

Enlarging a Base Line.—Where the base line is much smaller than the average length of the sides of the triangles it is necessary to enlarge it by triangulation. The method is illustrated in Fig. 166. AB is the measured base line. Stations C and D are chosen, one on either side of AB, and so disposed as to form well-conditioned triangles with the points A and B. The angles of the triangles having been measured the lengths of the sides may be calculated and also the length of the line CD, so that CD may be used as a base. CD may be about twice the length of AB, and may be expanded in a similar manner by arranging triangles on either side of it. The following example shows the method of calculating the length of CD, the particulars of the triangles being as entered in the first column of the succeeding table. It is only necessary to calculate the length of one side of each triangle

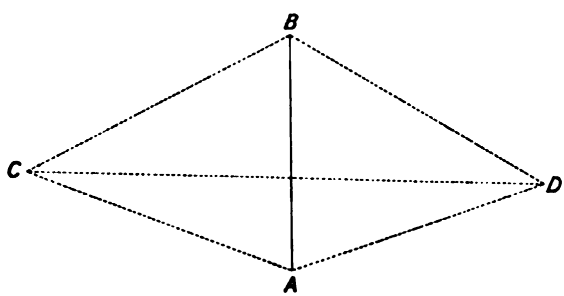

FIG. 166.—Enlarging a Base Line.

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in order to arrive at the length of CD, but it is better to calculate all four sides, as this furnishes a check on the accuracy of the results.

Calculation of Sides.

Triangle.	Logs and Log Sines.		Logs of Sides.
Triangle ABC.			
AB = 2873·7 ft.	3·45844		
A = 69° 14′	—	9·97083	3·56315 = log BC
C = 47° 17′	9·86612	3·59232	
B = 63° 29′	—	9·95173	3·54405 = log AC
Triangle ABD.			
AB = 2873·7 ft.	3·45844		
A = 72° 35′	—	9·97962	3·56529 = log BD
D = 48° 15′	9·87277	3·58567	
B = 59° 10′	—	9·93382	3·51949 = log AD

Calculation of Latitudes and Departures.
Line AB taken as Axis.

Line and Bearing.	Log Length and Log Sine.	Departure.	Log Length and Log Cosine.	Latitude.
AC	3·54405		3·54405	
69° 14′	9·97083		9·54969	
	3·51488	3272·5	3·09374	1240·9
AD	3·51949		3·51949	
72° 35′	9·97962		9·47613	
	3·49911	3155·8	2·99562	990·0
	Dep. diff.	= 6428·3	Lat. diff.	= 250·9
BC	3·56315		3·56315	
63° 29′	9·95173		9·64978	
	3·51488	3272·5	3·21293	1632·8
BD	3·56529		3·56529	
59° 10′	9·93382		9·70973	
	3·49911	3155·8	3·27502	1883·7
	Dep. diff.	= 6428·3	Lat. diff.	= 250·9

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The table of latitudes and departures brings out a total departure difference between the points C and D of 6428·3 ft. and a total latitude difference of 250·9 ft. The length of CD may then be calculated from the formula $CD^2 = \sqrt{6428{\cdot}3^2 + 250{\cdot}9^2}$, which gives CD = 6433·3 ft.

The second method of finding the length of a line from its latitude and departure given under traverse problems (p. 212) is not quite so laborious as the above method. The calculation by means of logarithms is as follows :

$$\begin{aligned}
\text{Log lat.} &= 2{\cdot}39950 \\
\text{Log dep.} &= 3{\cdot}80810 \\
\hline
\text{Diff.} &= 8{\cdot}59140 = \log\tan 2^\circ\ 14'\ 7'' \\
\text{Log dep.} &= 3{\cdot}80810 \\
\text{Log}\cos 2^\circ\ 14'\ 7'' &= 9{\cdot}99967 \\
\hline
\text{Diff.} &= 3{\cdot}80843 = \log 6433{\cdot}3
\end{aligned}$$

The length of the expanded base CD is, therefore, 6433·3 ft.

Selecting Triangulation Stations.—The facility with which a good triangulation system may be arranged depends very much on the nature of the country. Open undulating ground presents but little difficulty. The opposite is the case in flat wooded territory where the erection of special towers may be necessary in order to obtain sights over the top of the trees.

A general examination of the whole area should be made in the first place, special attention being paid to those heights and ridges which afford a wide view. Any eminence from which an extensive view can be obtained all round is sure to block the view in certain directions from points at a lower elevation. The higher points of the ground are, therefore, the most eminently suited to the purpose of observation stations and the preliminary examination will at once enable certain points to be fixed, which from their commanding position must be used as triangulation stations. Careful examination should then be made from those heights with the view to fixing probable positions for other stations. These positions will in turn be inspected in order to determine the conditions of view with respect to the proposed surrounding stations. As it is

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not easy to judge distances and shapes of triangles accurately by the eye an existing map of the area, even if somewhat rough, will be of great assistance in enabling a good arrangement of triangles to be arrived at with the minimum of trouble. The proposed positions of stations having been roughly marked on the map and the sides of triangles joined up it can be seen at a glance whether the arrangement is good or whether the shape of certain of the triangles ought to be improved.

When the ground has been carefully gone over and provisional positions have been fixed for many of the commanding stations and a good general idea of the possibilities of the area for the purpose of triangulation has been obtained, the laying out of the complete triangulation system will be proceeded with. A commencement would preferably be made at the base line, the triangulation network being gradually extended to cover the whole area without leaving any gaps or omissions. Where an existing map is available the triangles will be laid down on it as they are set out. It will sometimes be found that the laying out of a portion of the work in the manner judged to be best at the time will give rise to badly conditioned triangles further on, which can only be corrected by going back and altering some of the work already set out. For this reason it is desirable to keep the observation of the angles considerably in the rear of the setting out.

Where no existing map is at hand it will be necessary to plot the triangles roughly to a small scale as the setting out proceeds, in order that their shape may be seen at a glance. To enable this to be done the bearings of the lines may be roughly taken with a prismatic compass or with the compass of the theodolite. The plotting will be effected by the use of a protractor, starting from the base line or from any side whose length has been roughly determined. The employment of a small range-finder in conjunction with a theodolite or compass will permit of stations being plotted roughly from a single bearing and distance.

For picking out the positions of poles and signals at distances of from half a mile to a mile the use of a pair of binoculars is almost indispensable.

A common source of error in the angular work arises when the view to a station is obstructed so that the sight must be taken to a point on the pole or signal at some height above the ground. If

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the pole is not standing vertical the point observed may not be directly above the station, and an error in its position may result. It is, therefore, desirable to avoid fixing stations in places where tall signals are necessary, and the surveyor may have to decide whether in certain cases it would not be expedient to arrange somewhat poorer conditioned triangles in order to avoid such stations. As the accuracy of a triangulation survey is largely dependent on a proper arrangement of the network and on accurate sighting to the station points, it will be profitable to expend a considerable amount of care and judgment on the proper selection and marking of the latter.

Marking the Stations.—On a small survey the positions of the stations will usually be marked by wooden pegs or iron pins, according to the hardness of the ground. The positions of pegs should be very carefully referred by accurate measurements to definite points or marks on permanent objects, so as to be easily recoverable when required. Stone blocks with a mark cut on their upper surface, or concrete blocks, in which an iron pin or nail has been set may be used as permanent marks. In cultivated ground these would be set at such a depth in the ground that their tops would be beneath the reach of the plough.

As temporary sighting marks for the theodolite the ordinary 6-ft. ranging rods should be used, furnished with a small, bright coloured triangular flag at the top to enable their positions to be more easily picked out. Distant poles are more rapidly discovered by the binoculars than by the theodolite, as the field of view of the latter is small. Some poles 10 or 12 ft. long may be useful at stations where the view is obstructed near the ground. An assistant should be in attendance at such high signals to see that the pole is kept vertical while sights are being taken to it.

An important matter in connection with the reading of the angles where the observer at the instrument requires to direct several assistants at distances of half a mile and more is the arranging of a simple and workable set of communication signals. The outstretched arm and a white handkerchief may serve for simple semaphore signals. The number of signals should be confined to the minimum that will serve the purpose. It is necessary to have a signal to indicate to an assistant when he may lift the pole,

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another to indicate when he is to come in to the observer at the theodolite, and another to indicate when he is to proceed to the next station. A source of confusion, annoyance, and delay arises when a signal meant for one assistant is taken up by another and it is, therefore, very necessary to take such precautions as will ensure that a given signal will be accepted only by the person for whom it is intended.

By the exercise of a little foresight and prearrangement a programme of the day's operations may be drawn up which will enable definite duties to be assigned to the assistants and definite instructions imparted for their guidance, and the necessity for signalling may thereby be very much diminished.

Measurement of Angles.—In triangulation the consistency of the work depends entirely on the accuracy of the angular measurements and it is, therefore, of importance to take such precautions as will ensure that the best use is made of the instrument available and the time at the surveyor's disposal. The procedure in measuring an angle will usually vary, according as the angle stands by itself or occurs as one of a group about a station. The method of "repetition," after described, is appropriate to the measurement of a single angle, while the method known as "series" is more suitable for the measurement of the angles of a group.

The principal precautions which may be taken in order to attain the best results in reading angles with the theodolite are the following:—

(*a*) Read both verniers, or if there are three read them all. One of the verniers, having a distinguishing mark, will be used as the principal index and its reading will be recorded in full. Only the minutes and seconds of the other or others need be recorded in order to obtain a mean reading.

(*b*) Read the angle with the telescope normal and then with it inverted and take the mean of the two readings. If the angle is repeated several times the same number of observations should be made with the telescope normal as with it inverted.

(*c*) Read the angle on different portions of the graduated circle by setting the vernier index successively to different parts of the arc. If there are two verniers the index may conveniently be set to zero for the first angle. The reading of both verniers will give two determinations of the angle taken on opposite portions of the

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circle. If the index is then set to 90° and the angle again read on both verniers a total of four determinations at equal intervals round the circle will have been obtained. If six determinations equally spaced are desired the index may be set successively to zero, 60° and 120°. With a three-vernier theodolite six readings equally spaced will be obtained by setting the index to zero and then to 60°, or by setting to zero and then to 180°, or by setting successively to any two readings which are either 60° or 180° apart.

(*d*) Read the angles first clockwise and then anti-clockwise and take the mean.

Precautions (*a*) and (*c*) are directed towards eliminating errors due to imperfect graduation and centering of the circles.

Precaution (*b*) gets rid of errors due to imperfect adjustment of the line of collimation and of the horizontal axis of the telescope.

Precaution (*d*) tends to eliminate errors due to personal bias in observation and to slip of tangent screws. It is not very commonly employed in the class of survey in question.

Repetition Method.—With the telescope normal and starting with the index at zero read and record the value of the single angle. With the vernier plate clamped at this reading, loosen the lower clamp, sight back on the first station, fix the lower clamp and again turn the telescope through the angle. The circle reading now gives the double of the angle. Repeat the operation a third time with telescope normal and then make three more repetitions with the telescope inverted. By this process the angle has been added up on the circle six times, and one-sixth of the final reading will be accepted as a more precise determination than the single reading. Of course 360° must be added to the final reading for each complete revolution. By taking half the number with the telescope normal and the other half with it inverted errors due to incorrect adjustment of the line of collimation and of the horizontal axis of the telescope are eliminated.

A development of the above method which tends to still greater accuracy in the reading of angles and which may be adopted when extreme precision is essential consists in also measuring the explement of the angle by means of six repetitions, three with the telescope normal and three with it inverted. The angle and its explement should together amount to 360°, but there will usually be a small

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discrepancy. Apply half the discrepancy as a correction to each angle so as to make them sum to 360° and the angles so obtained may be accepted as the most probably accurate values.

Series Method.—The method known as "series," or "reiteration," is suitable for the measurement of the angles of a group having a common vertex point, and is therefore adapted to most of the work in a minor triangulation.

The method applied to the reading of the angles about the point O in Fig. 167, assuming that one vernier only was used, would be as follows: Set up the instrument at O, set the index to zero and sight the telescope on one of the stations, say, A. Then with the lower clamp fixed, loosen the upper clamp, rotate the telescope clockwise, sight successively on stations B, C, and D, reading the vernier in each case, and finally sight back on A and read the vernier. If the final reading differs by more than one minute from the starting reading discard the whole set. If the closing error is within one minute, assume that it has been introduced gradually and apply an equal correction cumulatively to each angle so as to eliminate the closing error. Make a second set of observations starting again with the telescope sighted on station A and with the vernier index set this time to 90°. If the telescope was normal for the first set invert it for the second set.

The readings of the various stations and the method of correcting the values and obtaining the mean angles are shown in the following table:—

READING ANGLES BY SERIES METHOD.

Line.	Readings.		Mean Reading.	Correction.	Corrected Reading.	Included Angle.
	Normal.	Inverted.				
OA	0° 0′ 0″	90° 0′ 0″	0° 0′ 0″	—	0° 0′ 0″	74° 36′ 11″
OB	74° 36′ 0″	164° 36′ 0″	74° 36′ 0″	+ 11″	74° 36′ 11″	88° 4′ 56″
OC	162° 40′ 30″	252° 41′ 0″	162° 40′ 45″	+ 22″	162° 41′ 7″	118° 42′ 41″
OD	281° 23′ 0″	11° 23′ 30″	281° 23′ 15″	+ 33″	281° 23′ 48″	78° 36′ 12″
OA	359° 59′ 0″	89° 59′ 30″	359° 59′ 15″	+ 45″	360° 0′ 0″	
			Error 45″		Total	360° 0′ 0″

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As the readings of the second set are booked they should be compared with the corresponding values of the first set as a check against large mistakes. The difference should be 90° in each case to within one minute. If the correspondence is satisfactory, make out a column of mean readings, taking the average for each line as regards the minutes and seconds only and not as regards the degrees which are written down as observed in the first set. The closing error of the mean readings is found to be 45 seconds, that is the final mean reading of the line OA differs by 45 seconds from the starting reading. A cumulative correction of 11¼ seconds should therefore be applied to each reading except the first, that is the readings of OB, OC, OD, and the final reading of OA should be increased by 11, 22, 33 and 45 seconds respectively. The column of corrected mean readings is then made out and the included angles are obtained by successive subtraction. It should be noted that while the angles so obtained have an appearance of being accurate to single seconds and fulfil the condition of summing correctly to 360° the individual angles may not be accurate to within ten seconds or more.

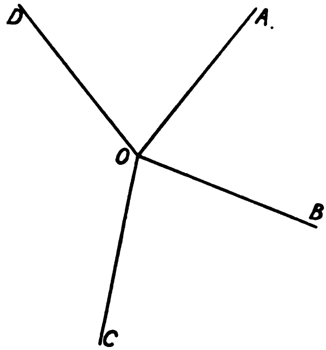

FIG. 167.—Series Method of Reading Angles.

When a theodolite with two verniers reading directly to twenty seconds and by estimation to ten seconds is employed, two sets of readings are taken as before, one set with telescope normal and the other with telescope inverted. Both verniers are read so that four values are obtained for the reading of each line. The complete reading is booked for vernier A in each case and only the minutes and seconds for vernier B. The mean reading of each line is, as regards the minutes and seconds, the average of the four observed readings, while the whole degrees are as read on vernier A in the first set. The correction of the readings and calculation of the included angles are made as in the example already given above.

Adjustment of the Angles.—When the angles have all been observed and their values obtained in the manners already described,

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the condition of affairs will be this: The angles round any station will sum to 360° but the three angles of any triangle may not add up exactly to 180°. Before calculation of the sides is commenced the angles of each triangle must be adjusted so as to sum exactly to 180°, without violating the condition as to summation of the angles round each station. When the adjustment of the angles has been completed the calculation of the sides may be proceeded with according to the convenient tabular form shown on p. 189.

Plotting Triangulation Stations.—When the lengths of the sides of the triangles have been computed the stations might be plotted on paper by intersecting arcs in the manner described for the triangles of a chain survey. This method is, however, hardly accurate enough for an important triangulation, and considerable difficulties would evidently arise if the work required to be distributed over many separate sheets. The co-ordinate method is by far the most suitable for the plotting of the stations of a triangulation. The first requirement is to fix the directions of the co-ordinate axes, which will to some extent govern the arrangement of the sheets as the edges of these will for convenience be taken parallel to the co-ordinate axes. In general it is preferable to adopt true north as the direction of one of the axes. The second requirement is to determine the bearing of one of the survey lines, near the centre of the area if convenient, with reference to the north and south axis. The triangulation may then, for the purpose of calculating bearings, be assumed to be divided up into a series of closed traverse polygons, arranged to include all the stations. To illustrate this Fig. 168 shows a small triangulation, and Fig. 169 shows an arrangement of polygons suitable for the calculation of bearings and co-ordinates. Line AB represents the survey line whose bearing has been first determined. When the angles of all the triangles

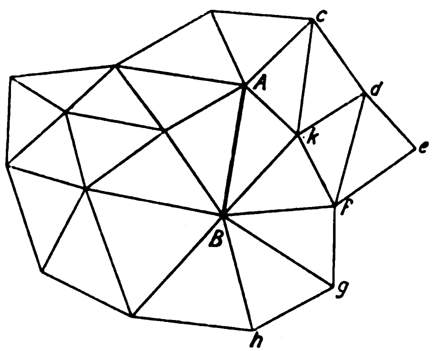

FIG. 168.—Arrangement of Small Triangulation.

are known the values of the interior angles of the polygon A*cdefgh*B can evidently be easily found, and hence the whole circle bearings of the sides of the polygon can be determined by the method given on p. 147 in connection with traverse surveying, and the reduced bearings for purposes of calculating latitudes and departures can also be found in the manner already described. In fact, when the bearings of the sides have been calculated, the remainder of the work of computation and plotting is exactly the same as for a traverse survey.

Errors in Reading Angles.—The principal sources of error in reading angles which arise in connection with the use of the theodolite are the following:—

(*a*) Theodolite not set exactly over the station mark.

(*b*) Theodolite not correctly levelled.

(*c*) Telescope not correctly focussed.

(*d*) Incorrect bisection of the station signal sighted to.

(*e*) Station signal displaced from its true position, or not vertical.

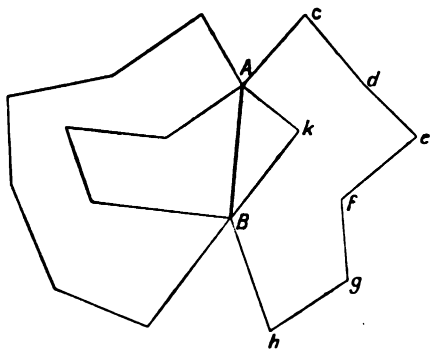

FIG. 169.—Polygons for Plotting Small Triangulation.

(*f*) Natural causes, including wind, heat, irregular refraction of the atmosphere, soft ground, frozen ground, &c.

The instrument itself may give rise to error in the angular work from the following causes:—

(*g*) Incorrect adjustment of the line of collimation of the telescope with respect to its horizontal axis.

(*h*) Incorrect adjustment of the horizontal axis of the telescope.

(*i*) Inaccurate graduation and centering of the circle and verniers.

Apart from the above sources of error the observer may make mistakes, such as the turning of the wrong tangent screw or the misreading of a vernier.

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Error in Planting the Theodolite.—This error can with little trouble be made almost inappreciable in minor triangulation. Suppose that the triangulation stations were at distances of 3,400 ft. apart (this would not be an inappropriate average length for a minor triangulation), then by setting up the theodolite 1 ft. away from the true position of a station a maximum error of one minute might be introduced in an angle of 60°, and of 1·7 minutes in an angle of 120°, and if the theodolite were set up 1 in. away from the station the maximum resulting errors in angles of 60° and 120° would be five seconds and eight seconds respectively. These latter amounts cannot be measured directly with the ordinary 5-in. or 6-in. theodolite. As it is easy to set up within $\frac{1}{2}$ in. of the true mark without waste of time the error due to incorrect planting should be almost inappreciable, and it will evidently serve no useful purpose to spend time in adjusting the plumb-bob to the last $\frac{1}{16}$ in. The shorter the length of the lines the more accurate does the setting require to be.

Error from Incorrect Levelling.—If the instrument is in adjustment and the plate level bubbles are brought as nearly as may be to the centre of their runs the error in measuring a horizontal angle due to the plates not being exactly level would be almost negligible. If the graduated circle is not level and the vertical axis in consequence not vertical the angle instead of being measured in a horizontal plane will be measured in an inclined plane, and the result will be different. If the points sighted are at the same elevation, a slight inclination of the graduated circle will not have much effect on the angles read, but if the telescope requires to be tilted up and down to sight on high and low points the angles read may be much in error. For very important work the final levelling instead of being done with the small plate levels should be effected by using the large telescope level as this is much more accurate than the others.

Error from Incorrect Focussing.—If the eyepiece is not correctly focussed on the cross hairs the line of sight through the telescope will be variable according to the position of the eye. If the eye is moved from side to side the cross hairs will appear to move relative to the object. Under such conditions the angular work is unreliable,

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as the position of the eye may be different for successive sights. The procedure to obtain proper focussing has been described on p. 127.

Error from Incorrect Bisection.—If the line of sight, instead of striking the centre of a signal, strikes at 1 in. to the side of the centre the resulting error will be about five seconds if the distance is 3,400 ft. If the signals are ordinary ranging rods, then provided the centre of the cross hairs appears on the rod the possible angular error can only be two or three seconds. The difficulty with ordinary instruments when the sights run to 5,000 ft. is that the apparent thickness of the hairs becomes comparable with the thickness of the signal, and it is, therefore, difficult to determine when the bisection is exact, but there is no tendency for the angular error to increase on this account when the sights are long. Special precautions should be taken to avoid bisection errors when the sights are short.

Error from Displacement of the Signal.—This may occur if the stations are marked by pegs and ranging rods planted beside the pegs are used as sighting signals. If the rod is planted vertical and on the line from the theodolite through the centre of the peg no error will occur, but if the pole is to one side of the correct line an error will result. This will most readily happen if a rod which was in correct position for a certain sight is inadvertently left in the same position for some other sight.

The greatest possibility of error occurs when a tall pole has to be erected as a signal in order that a station may be visible from certain others. If a sight is taken to a point near the top of a pole which is not vertical the point sighted will not be directly above the true station mark, and hence the angle read may not be correct. If only a short bit of the top of a pole is visible in the telescope the observer may not be able to tell if the pole is vertical, and hence great care should be taken to ensure that all signals which must be observed at a point above the ground are kept truly vertical during observation. If a signal pole requires to be specially tall it should be plumbed in two directions with the theodolite and stayed with guy-ropes so as to retain the vertical position.

Errors from Natural Causes.—A high wind may cause the theodolite to shake and vibrate, and render accurate work impossible.

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The reading of angles in important work should, therefore, be confined to fairly calm weather.

A strong sun beating on one side of the telescope may cause such a difference in temperature between the two sides as to give rise to bending of the tube. This would affect the line of sight and, if it happened between the readings of two successive lines, an error would be introduced into the angle. The precaution may be adopted of shading the instrument from the sun, and it will be desirable to take the readings of a group as rapidly one after the other as possible, so that little time may be allowed for the temperature state of the instrument to change.

When the ground has been heated by a strong sun the layers of air near the ground are usually of very variable and changing temperatures, and objects viewed through such an atmosphere appear to waver and wriggle about. A similar effect in a lesser degree occurs near the ground at the summit of a ridge or hill when a wind is blowing over it. The above effects are due to irregular refraction of the atmosphere, and when such conditions obtain the reading of angles should not be attempted.

When the theodolite is set up on soft ground the treading of the observer may cause yielding of the ground and disturbance of the instrument. The observer should move as little as possible and should particularly avoid stepping near the legs of the instrument. If the ground is very soft the latter may require to be supported on three stakes driven till they are firm.

If the theodolite is set up on wet ground which has become frozen or on ice, the sharp iron-shod points of the legs will gradually settle down and the settlement may be irregular. The settlement is, however, slow, but, to avoid error in the angles, the observation of each group should be carried out as rapidly as possible.

Error from Incorrect Adjustment of Line of Collimation.—If the line of collimation or line of sight of the telescope is not at right angles to the horizontal axis it will not revolve in a plane, but will describe a flat cone on rotation. This will not give rise to error in the angles if the objects sighted are all at one angular elevation, but will give rise to very slight error if the objects are at different elevations. Error in an angle due to incorrect adjustment of the line of collimation will be completely eliminated by taking the mean

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of a number of observations, one-half of which are made with telescope normal and the other half with telescope inverted.

Error from Incorrect Adjustment of Horizontal Axis.—If, when, the horizontal plates have been accurately levelled, the horizontal axis of the telescope is not level the line of sight of the telescope will not revolve in a vertical plane, but in an inclined plane, and if the observed objects are at different angular elevations the angles obtained will be incorrect. This source of error is also eliminated by taking observations with telescope normal and with telescope inverted, as described in the preceding paragraph.

Error from Incorrect Graduation, &c.—This should be very small in a good instrument. The graduation of a circle may be tested by measuring a small angle by repetition until the whole circumference is traversed, and noting whether any difference is found between the successive angles. An occasional difference not exceeding the smallest direct reading of the vernier may be expected, and the difference between *any* two readings should not exceed this amount. Irregular differences, provided the observations were carefully made, would signify that the circle was poorly graduated. Graduation errors are partly eliminated by measuring the angle on various portions of the circle, as in the "series" method. This also partly gets rid of errors due to bad centering of the vernier plate with respect to the divided circle.

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CHAPTER XV

SOME SURVEY, TRAVERSE, AND TRIANGULATION PROBLEMS

Some examples of the methods of chain surveying applied to the solution of special problems were given in Chapter VII. Many of these problems can be much more readily and simply solved by the use of the theodolite. Some typical problems are dealt with in this chapter, mainly for the purpose of educing useful ideas as to the methods by which problems of a similar nature may be attacked. The problems considered include: setting out a perpendicular from an accessible or inaccessible point to a given line; setting out a line parallel to a given line; running a straight line between two points when an obstacle intervenes; finding distance to an inaccessible point and distance between two inaccessible points; traverse problems; three-point problem; determination of heights by the theodolite; trigonometric levelling.

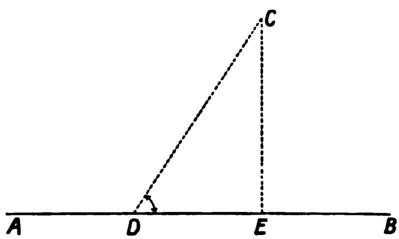

Fig. 170.—Perpendicular from a Point to a Line.

Perpendiculars.—To set out a perpendicular from the given point C to the given line AB (Fig. 170). Set up the theodolite at a convenient point D on the line AB and measure the angle CDB. Next set up at C, sight on D, and lay off an angle DCE equal to 90° − CDB. Line CE given by the theodolite will be at right angles to AB, since the sum of the angles EDC and DCE is equal to 90°.

When the given point is inaccessible the method shown in Fig. 171 may be employed. H is the given point, FG is the given line. Choose suitable points F and G on the line and measure the angles HFG and HGF. Measure also the length FG. Let HK represent the perpendicular to the line FG. Calculate first the length of FH. In the triangle FHG we have—

Angle FHG = 180° − (HFG + HGF), or H = 180° − (F + G).

Also $\frac{FH}{\sin G} = \frac{FG}{\sin H}$ $\therefore FH = \frac{FG \sin G.}{\sin H}$

But $\frac{FK}{FH} = \cos F$ $\therefore FK = FH \cos F.$

So that $FK = \frac{FG \cos F \sin G}{\sin H}$

Point K will then be got on the ground by measuring off this calculated distance along the line from point F.

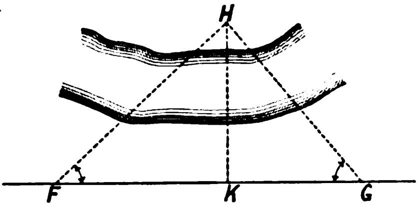

FIG. 171.—Perpendicular from an Inaccessible Point.

Setting out a Line parallel to a given Line.—In Fig. 172, AB is the given line, C is the given point. At a suitable point, D, on the line AB, measure the angle CDB. Set up the theodolite at C, sight on point D, and lay off the angle DCE equal to 180° − CDB. The theodolite will then point in a direction parallel to the line AB, and the required line CE may be ranged out.

Running a Line between two Points when an Obstacle Intervenes.—It is required to run a straight line between the two points A and B (Fig. 173). An obstacle intervenes so that one point cannot be seen from the other.

First method. Choose a point, such as C in the figure, from which both points A and B can be seen. Measure the distances AC and BC. Set up the theodolite at C and measure the angle ACB. Produce the lines AC and BC to E and D respectively, making CE = CA and CD = CB. Set up the theodolite at E and measure the angle CED. The triangles DCE and ACB are similar, so that the angle CAB is equal to the measured angle CED. The theodolite is, therefore, set up at A and the angle CAA′ is laid off equal to CED. Points may, therefore, be set in line up to A′. The angle CBA is equal

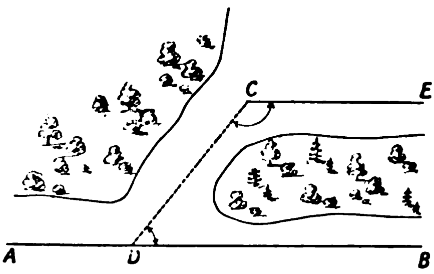

FIG. 172.—Setting out a Parallel Line.

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to 180° — (CAB + ACB) and this may, therefore, be laid off by the theodolite at point B, thus giving the portion of the required line BB′. The two portions AA′ and BB′ so set out will be in one straight line.

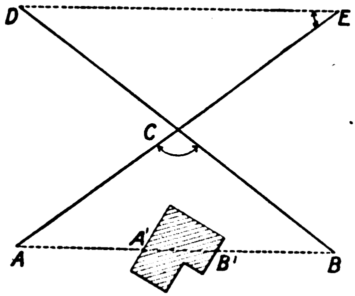

Fig. 173.—Running Straight Line between two Points.

Second method. In Fig. 174, F and G are the given points. A suitable point H is chosen as before and the lines FH and HG and the angle at H are measured. Then in the triangle FHG two sides and the included angle are known so that the angles HFG and HGF can be calculated by the formulas given on p. 354. They may then be set off at the points F and G respectively so as to give the portions of the required line on either side of the obstacle.

Otherwise let GK be a perpendicular to the line FH produced.

$$\text{Then angle KHG} = \theta = 180° - \text{FHG}.$$
$$\text{HK} = \text{HG} \cos\theta. \quad \text{KG} = \text{HG} \sin\theta.$$
$$\text{Tan HFG} = \frac{\text{KG}}{\text{KF}} = \frac{\text{HG} \sin\theta}{\text{FH} + \text{HG} \cos\theta}.$$

The angle HFG corresponding to the above value of the tangent can be found from trigonometrical tables. The angle HGF can then be deduced.

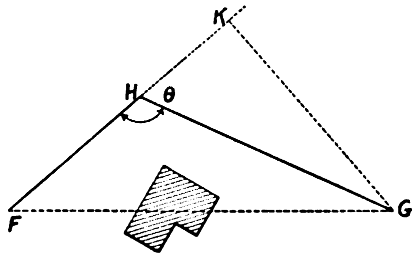

Fig. 174.—Running Straight Line between two Points.

Distance to an Inaccessible Point.—It is required to find the distance between the points A and B (Figs. 175 and 176), point B being inaccessible. Any convenient base line AC may be set out, as in Fig. 175. If the length of AC is measured and also the angles BAC and ACB, then the length of the side AB can be calculated by the method given in the chapter on triangulation.

When a suitable base line can be set out at right angles to ABt he work of calculation is simplified. In Fig. 176, AD is set out at right

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angles to AB and its length is measured. The angle ADB is measured.

Then AB = AD tan ADB.

Distance between two Inaccessible Points.—A and B (Fig. 177) represent the two inaccessible points. Measure off a suitable base

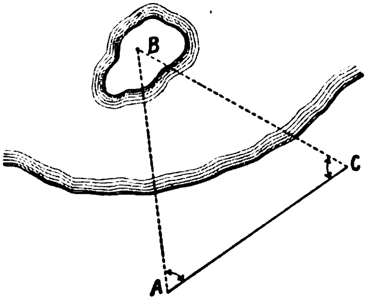

FIG. 175.

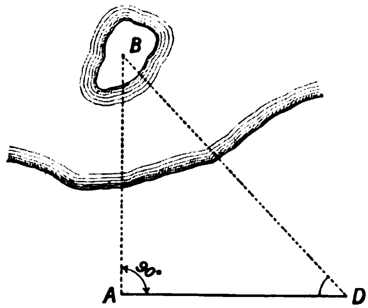

FIG. 176.

Distance to Inaccessible Point.

line CD. Measure the angles ACD, BCD, ADC, BDC. Then in each of the triangles CAD and CBD one side and the two adjacent angles are known so that the remaining sides may be calculated by the methods of triangulation. Calculate side CA of the triangle CAD, and side CB of the triangle CBD. Then in the triangle ACB the two sides CA and CB are known and the included angle ACB is also known, so that the length of the side AB can be calculated by the formula appropriate to the case, as given on p. 354.

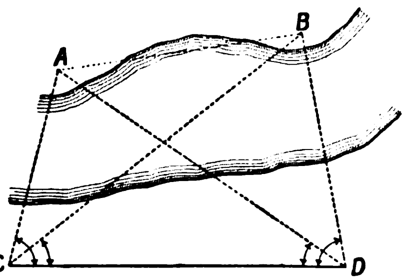

FIG. 177.—Distance between Inaccessible Points.

Distance of a Boat from the Shore.—The following method of fixing the positions of points on the water is used in taking soundings. The direction of a line of soundings is marked by two poles A and D erected on shore (Fig. 178). The base line AC is measured off at right angles to AD. The boat is rowed out keeping in line with the poles A and D, and the positions of soundings are fixed by an observer in the boat

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measuring with a sextant the angle made with the points A and C. Thus, to fix the position of the boat when at B the observer measures the angle ABC.

$$\text{Then } \frac{AC}{AB} = \tan ABC, \text{ or } AB = \frac{AC}{\tan ABC}$$
$$\text{or } AB = AC \cot ABC.$$

Given the Latitude and Departure of a Line to find its Length and Bearing.—The graphical solution of this problem is simple. Plot the departure *ab* to scale (Fig. 179) and lay off the latitude *bc* at right angles to it. The length of the line will be got by scaling *ac*, and its bearing with reference to the meridian will be obtained by measuring the angle *a* at point *c* with the protractor. The angle *a* so obtained is the reduced bearing of the line, and the whole circle bearing will be got by the converse of the process shown on p. 169, attention being paid to the signs of the latitude and departure.

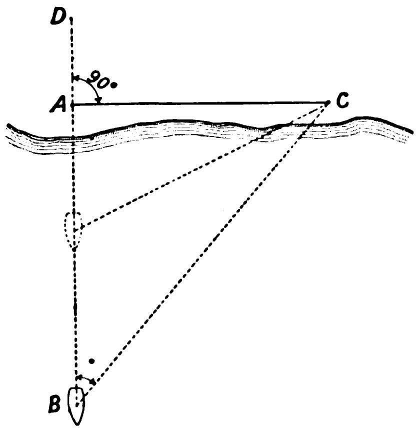

FIG. 178.—Distance of Boat from Point on Shore.

By calculation the length of the line is obtained from the formula

$$ac = \sqrt{ab^2 + bc^2} = \sqrt{\text{dep.}^2 + \text{lat.}^2}.$$

The angle *a* is obtained from the formula

$$\tan a = \frac{ab}{bc} = \frac{\text{departure}}{\text{latitude}}.$$

When the angle *a* has been determined by the above method the length of the line can be obtained from the formula

$$ac = \frac{ab}{\sin a} = \frac{\text{departure}}{\sin a} \text{ and also } = \frac{bc}{\cos a} = \frac{\text{latitude}}{\cos a}.$$

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The calculation of the length by means of this formula is less laborious than by the first formula given above.

Given the Co-ordinates of Two Points to find the Length and Bearing of the Line joining them.—Find the latitude and departure of the line and use the formulas given in the preceding paragraph. The latitude is equal to the difference of the two meridian co-ordinates treating them algebraically, and the departure is similarly equal to the difference of the two co-ordinates in the perpendicular direction.

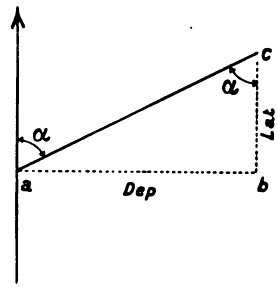

FIG. 179. — Length of a Line from Latitude and Departure.

Example. The co-ordinate distances of two points *a* and *c* are (N. 537 ; E. 421) and (N. 220 ; W. 209) respectively. It is required to find the length and bearing of the line *ac* joining the points.

$$\text{Latitude of line} = 537 - 220 = 317$$
$$\text{Departure of line} = 421 + 209 = 630$$

$$\text{Length of line} = \sqrt{\text{dep.}^2 + \text{lat.}^2} = \sqrt{630^2 + 317^2} = 705{\cdot}3 \text{ feet.}$$

$$\text{Tan } a = \frac{\text{departure}}{\text{latitude}} = \frac{630}{317} = 1{\cdot}9874.$$

From which angle $a = 63° \, 17'$. The line *ac* is situated with respect to the co-ordinate axes, as shown in Fig. 180.

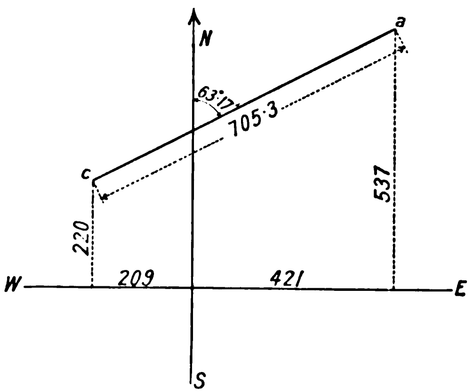

FIG. 180.—Length of Line from Co-ordinates.

To find the Length and Bearing of an Omitted Side of a Traverse Polygon.—The lengths and bearings of all the sides of a polygon except one having been measured it is required to calculate the length and bearing of the omitted side. The omitted side may be considered as a large closing error. Work out the latitudes and

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departures of all the measured lines. Find the difference between the sum of the north latitudes and the sum of the south latitudes. This will be the latitude of the omitted side. Similarly find the difference between the sums of the east and west departures to obtain the departure of the omitted side. From its latitude and departure so found calculate the length and bearing of the omitted side using the formulas already given, in the manner shown in the example which has just been worked out.

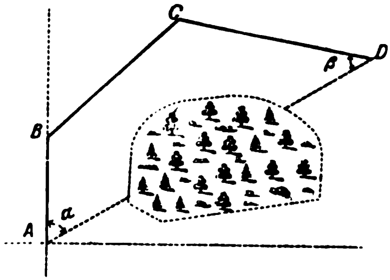

FIG. 181.—Ranging a Straight Line.

This problem may be practically applied in the ranging out of a straight line between two points when a large obstruction, such as a thick wood, intervenes. A system of traverse lines is run around the obstruction to connect the points and the bearings and lengths of the lines are measured. Referring to Fig. 181, A and D are the points between which the straight line is to be ranged. ABCD is the traverse connecting the points. Line AB is taken as the meridian direction, A being the origin of co-ordinates. The latitudes and departures of the lines AB, BC, CD are calculated and their algebraic summations give the co-ordinates of point D and the latitude and departure of the line AD. From these the angle α and the length of the line AD can be calculated. The angle β can also be found by taking the difference of the bearings of the lines CD and AD. By setting off the angles α and β at the points A and D respectively portions of the straight line may be set out and extended as the obstruction is cleared away.

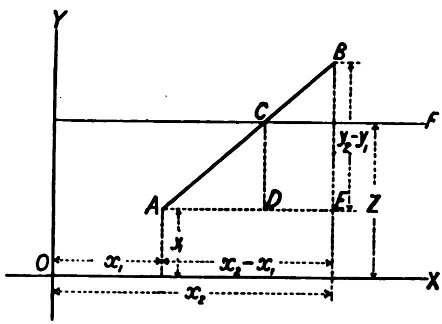

FIG. 182.—Survey Line crossing a Sheet Boundary.

To find where a Survey Line cuts a Parallel to one of the Axes.—This problem occurs where a survey is plotted on separate sheets.

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Some of the survey lines will cross the boundary lines of the sheets and it is necessary to find the points of crossing. The problem is illustrated in Fig. 182. OX and OY are the co-ordinate axes. Points A and B are the extremities of the survey line, and their co-ordinates are x_1y_1 x_2y_2 respectively. Line CF represents the common joining line of the sheets, parallel to the axis OX and at distance Z from it.

The departure of the line AB $= x_2 - x_1$ and its latitude $= y_2 - y_1$. The co-ordinates of point C will be found by determining the distance AD.

$$CD = Z - y_1.$$

From similar triangles $\frac{AD}{DC} = \frac{AE}{EB}$

or $$AD = \frac{DC \times AE}{EB} = \frac{(Z - y_1) \times (x_2 - x_1)}{(y_2 - y_1)}.$$

The X— co-ordinate of point C is equal to

$$x_1 + AD, \text{ or } = x_1 + \frac{(Z - y_1) \times (x_2 - x_1)}{(y_2 - y_1)}.$$

The method is similar when the joining line is parallel to the Y-axis, the Y-co-ordinate being then equal to

$$y_1 + \frac{(Z - x_1)(y_2 - y_1)}{(x_2 - x_1)}.$$

Example. The co-ordinates of the stations at the ends of a survey line are 1539·2 E., 268·3 N., and 2275·4 E., 523·7 S. To find where the survey line crosses a sheet boundary which is parallel to the meridian and at a distance of 2000·0 east of the origin :

$$\begin{array}{rlrlrl} x_2 = & 2275{\cdot}4 & y_2 = & -\ 523{\cdot}7 & Z = & 2000{\cdot}0 \\ x_1 = & 1539{\cdot}2 & y_1 = & 268{\cdot}3 & x_1 = & 1539{\cdot}2 \\ \hline x_2 - x_1 = & 736{\cdot}2 & y_2 - y_1 = & -\ 792{\cdot}0 & Z - x_1 = & 460{\cdot}8 \end{array}$$

Applying these figures in the formula given above the Y-co-ordinate of the point required will be

$$268{\cdot}3 + \frac{460{\cdot}8 \times -792{\cdot}0}{736{\cdot}2} = 268{\cdot}3 - 495{\cdot}8 = -227{\cdot}5.$$

The minus sign indicates a south co-ordinate, so that the survey

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line will cross the joining line of the sheets at a distance of 227·5 below the X-axis.

Two Omitted Measurements in a Polygon.—The co-ordinates of a polygon can be calculated, provided that not more than two of the linear and angular measurements have been omitted. The problem has been already solved for the case where the length and bearing of one side have been omitted. We shall now deal with the cases where two adjacent sides are involved. The omitted measurements may then be as follows :—

(*a*) The lengths of two sides.

(*b*) The bearings of two sides.

(*c*) The length of one side and the bearing of another.

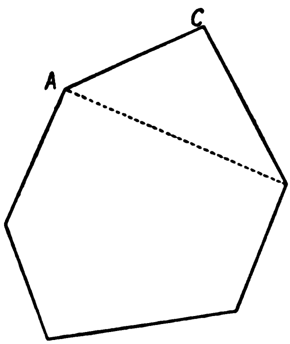

FIG. 183.—Two Omitted Measurements.

In Fig. 183 let AC and BC be the sides to which the omitted measurements refer. The general method of procedure in each case is to ignore in the first place the sides AC and BC, and to consider AB as a closing side of the polygon. The latitudes and departures of all the known sides of the polygon having been calculated, the length and bearing of the omitted side AB are

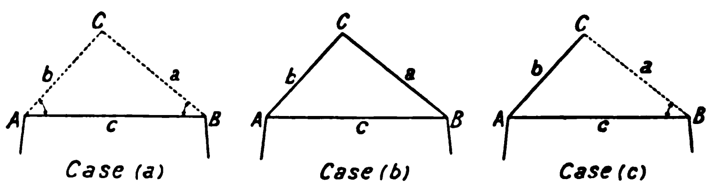

FIG. 184.—Two Omitted Measurements in a Polygon.

worked out in the manner already explained. Then in the triangle ABC, Fig. 184, the known quantities in the several cases are :

(*a*) The length of side AB and the angles at A and B. These angles require to be deduced from the known bearings of the lines AB, AC and BC.

(*b*) The lengths of the three sides.

(*c*) The lengths of two sides and the angle opposite one of them.

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This angle also requires to be deduced from the known bearings.

The three known quantities are sufficient in each case to enable the magnitudes of the remaining sides and angles of the triangle to be calculated. The necessary trigonometric formulæ for the solution of the triangles are given on p. 354.

Case (*a*). Side *c* and angles A and B are known. To find lengths of sides *a* and *b*—

Find first the angle C, which is equal to $180° - (A + B)$.

Then $$a = \frac{c \sin A}{\sin C}; \quad b = \frac{c \sin B}{\sin C}.$$

Case (*b*). Sides *a*, *b*, *c* are known. To find angles A and B

Let $$s = \frac{a + b + c}{2}.$$

Then $\sin \frac{A}{2} = \sqrt{\frac{(s-b)(s-c)}{bc}}$ and $\sin \frac{B}{2} = \sqrt{\frac{(s-a)(s-c)}{ac}}.$

These formulæ enable the angles A and B to be calculated and from them the required bearings of the sides AC and BC can be deduced.

Case (*c*). Sides *b* and *c* and angle B are known. To find side *a* and angle A

Find first the angle C from the formula $\sin C = \frac{c \sin B}{b}$.

Then angle $A = 180 - (B + C)$, and this enables the bearing of side AC to be determined.

Also $$\text{side } a = \frac{b \sin A}{\sin B}.$$

When the lengths and bearings have been calculated by the methods above explained, the latitudes and departures of the sides to which they refer may be computed and the calculation of the co-ordinates may be completed.

Three-point Problem.—It is sometimes useful to be able to fix the position of a point merely by angular observations taken at the point. This can be done by reading the angles made with three known points suitably situated, as shown in the three diagrams, (*a*), (*b*) and (*c*) (Fig. 185). In each case A, B, and C represent

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the known points, their relative positions being fixed by the known distances d and e and the known angle F. O is the point to be fixed, and to this end the angles D and E at point O are measured.

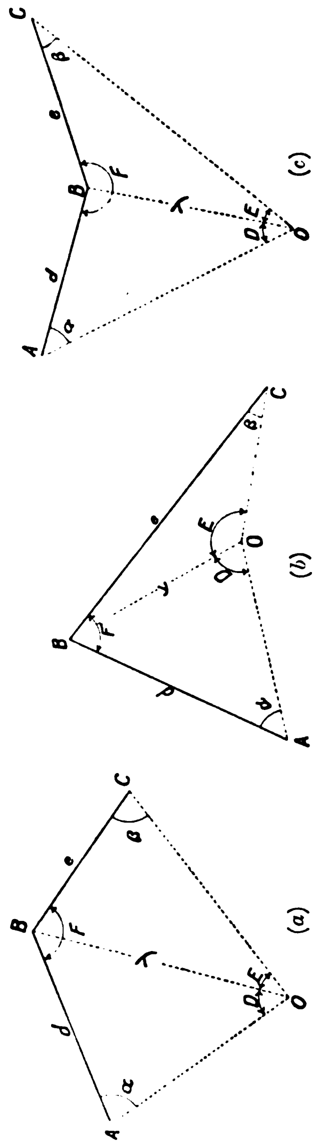

FIG. 185.—Three-point Problem.

The problem can be solved graphically by the use of tracing paper. From a point on the tracing paper draw three lines of indefinite length, making the angles D and E with each other so as to represent the observed lines OA, OB, and OC. Shift the paper about over the plotted points A, B, and C, until each line passes exactly through its point. The required point O may be then pricked through.

The problem may also be solved by trigonometrical methods, of which the simplest consists in calculating the angles α and β.

The four interior angles of the quadrilateral ABCO are together equal to 360°, so that if $S = \alpha + \beta$, $S = 360° - (D + E + F)$.

In triangle OAB we have

$$\frac{y}{\sin \alpha} = \frac{d}{\sin D} \text{ or } y = \frac{d \sin \alpha}{\sin D},$$

and in triangle OBC we have similarly $y = \dfrac{e \sin \beta}{\sin E}$,

so that $\dfrac{d \sin \alpha}{\sin D} = \dfrac{e \sin \beta}{\sin E}$,

$$\text{or } \frac{\sin \beta}{\sin \alpha} = \frac{d \sin E}{e \sin D} \quad . \quad . \quad (1)$$

Also $\beta = S - \alpha$, so that $\sin \beta = \sin S \cos \alpha - \cos S \sin \alpha$.

Substituting for $\sin \beta$ in (1) we get,

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$$\frac{\sin S \cos \alpha - \cos S \sin \alpha}{\sin \alpha} = \frac{d \sin E}{e \sin D},$$

or

$$\sin S \cot \alpha - \cos S = \frac{d \sin E}{e \sin D},$$

which gives

$$\cot \alpha = \frac{d \sin E}{e \sin D \sin S} + \cot S. \quad . \quad . \quad (2)$$

The value of cot α may be computed from the above formula, and the angle α will then be got from a table of cotangents. Angle β may then be got directly from the formula $\beta = S - \alpha$. Angle β may also be found from a formula corresponding to (2), namely :—
Cot $\beta =$

$$\frac{e \sin D}{d \sin E \sin S} + \cot S.$$

When the angles α and β have been calculated, the required point O will be given by the intersection of the lines, got by laying off these angles at the points A and C respectively.

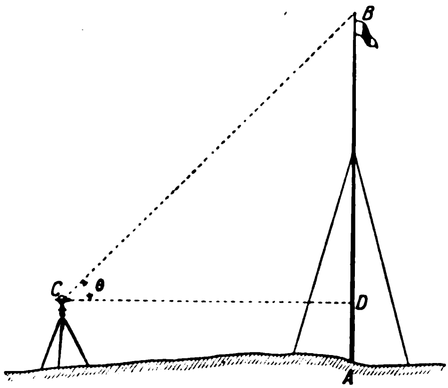

FIG. 186.—Height by Theodolite.

Determination of Heights by the Theodolite. — A simple case is illustrated in Fig. 186. The theodolite is planted at a measured distance CD from the vertical object AB, whose height is to be determined. A horizontal line of sight CD is given and point D is marked. The vertical angle DCB is measured. The height AD is measured. Then BD = CD tan θ, and the whole height AB = AD + CD tan θ. The height AD instead of being measured directly might be found similarly to DB by taking a sight to point A and measuring the angle ACD.

A more general case is shown in Fig. 187. The foot of the object cannot be seen from the instrument, and an inclined sight is therefore taken to a point E, and the height EA is measured. Then BD = CD tan α. ED = CD tan β. The whole height AB = BD — ED + EA, or AB = CD tan α — CD tan β + EA = CD (tan α — tan β) + EA.

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Where the projection of B on the ground is inaccessible, point C may be taken as one end of a base line from which point B is fixed by triangulation, and the distance CD may then be calculated.

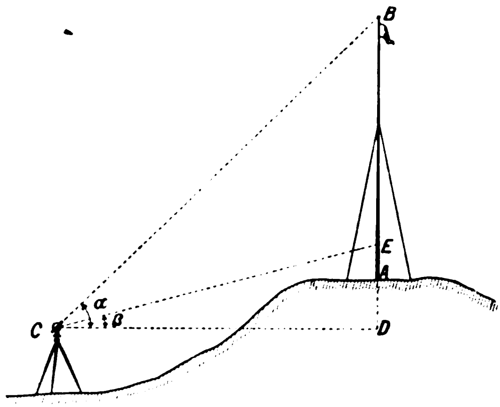

Fig. 187.—Height by Theodolite.

Trigonometric Levelling.—When the horizontal distance between two points is known the relative altitudes of the points may be determined by reading with the theodolite the vertical angle of elevation or depression which the one point makes with the other. This method of finding altitudes is known as trigonometric levelling. The spherical form of the earth must be taken into account, as it appreciably affects the results for all distances greater than about one furlong.

Curvature of the earth. In Fig. 188, ABC represents a line of uniform elevation on the surface of the earth, O is the centre of the earth, and AD is a tangent to the earth's surface at point A, that is, AD corresponds to the horizontal line of sight of a theodolite set up at A. Let the radial line OB be produced to meet line AD in point D. BD is vertical to the earth's surface at point B, and the height BD = h is the height above the level of point A at which the horizontal line through A strikes the vertical through B. Let earth's radius be r, distance

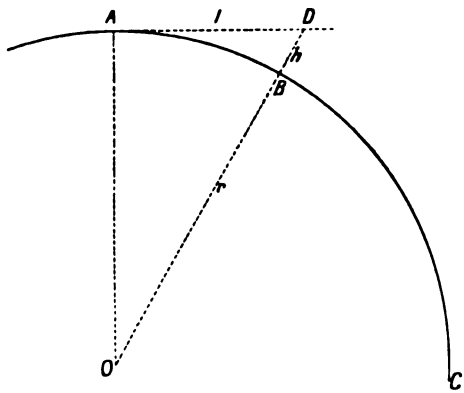

Fig. 188.—Curvature of the Earth.

AD be l. In triangle OAD, $OD^2 = OA^2 + AD^2$, or $(r + h)^2 = r^2 + l^2$, which gives $h = \frac{l^2}{2r + h}$.

For distances up to a few miles h is very small compared with $2r$ and h may with very little error be taken equal to $\frac{l^2}{2r}$.

The formula may be put in the form $h = C\ l^2$, where C is a constant depending on the units employed.

When h is in feet and l is in miles the formula is $h = 0{\cdot}667\ l^2$, so that when l is equal to one mile h is equal to 0·667 ft. or almost exactly 8 ins.

When h is in feet and l is also in feet the formula becomes

$$h = \frac{2{\cdot}39\ l^2}{10^8}, \text{ or } h = \frac{2{\cdot}39\ l^2}{100{,}000{,}000}.$$

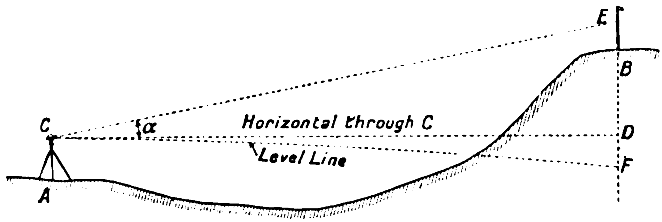

FIG. 189.—Finding Height of Point above the Theodolite.

The latter formula would be used when distances are given in feet, the former when distances are given in miles.

To find Difference of Altitude between two Points.—The trigonometric method applied to finding the difference of altitude between two given points is illustrated in Fig. 189. It is assumed that the points are not much more than one mile apart. A and B are the two given points. C is the centre of the telescope axis of the theodolite set up at A. EF is a vertical line through point B. CD represents a horizontal line of sight at point C, while curving line CF is a level line through point C. The theodolite is sighted to a point E on a signal erected at B. The vertical angle ECD is measured. Then, remembering that point F is at the same level as point C we get the total difference of elevation between points A and B = AC + FB = AC + FE − EB = AC − EB + FD + DE.

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But DE = CD tan a, and FD = $\frac{2 \cdot 39 \text{ CD}^2}{10^8}$.

Therefore height of B above A = AC − EB + CD tan $a + \frac{2 \cdot 39 \text{ CD}^2}{10^8}$.

The heights AC and EB must be separately measured.

Example. Centre of axis of telescope was 4·25 ft. above station mark A. Vertical angle of elevation read to point E was 2° 27′ 40″ and point E was 3·85 ft. above station mark B. To find height of B above A, the horizontal distance between the points being 4,381 ft.

Calculation for ED.	Calculation for FD.
Log 4381 = 3·64157 Log tan 2° 27′ 40″ = 8·63327 Log CD = 2·27484 CD = 188·3	Log 4381² = 7·28314 Log 2·39 = 0·37840 7·66154 Log 10⁸ = 8·00000 Log FD = $\bar{1}$·66154 FD = 0·4587

Total height of B above A = 4·25 — 3·85 + 188·30 + 0·46
= 189·16 ft.

In the above treatment of this problem it has been assumed that the vertical at B is parallel to the vertical at A, and that therefore

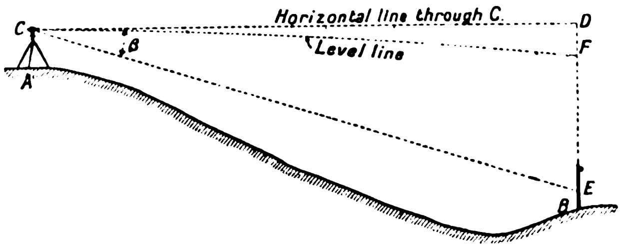

FIG. 190.—Finding Elevation of Point below the Theodolite.

the angle CDE is a right angle. Also the effect of refraction has been ignored. The angle CDE is always greater than a right angle (about one minute greater when the distance is one mile) in the case of Fig. 189, and less than a right angle in the case of Fig. 190.

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Refraction is generally allowed for by making a slight reduction of the curvature allowance. The error introduced by making the foregoing assumptions is negligible when the distance between points does not exceed one mile. The errors increase rapidly as the distance becomes greater and become important when the distance is several miles, but the exact solution applicable to such a case is beyond the scope of this book.

Fig. 190 illustrates the case when the observation is made from the higher of the two points. The difference of altitude is then equal to BF — AC = BE — AC + ED — DF,

$$\text{or} = \text{BE} - \text{AC} + \text{CD} \tan \beta - \frac{2{\cdot}39 \ \text{CD}^2}{10^8}.$$

Note that the allowance for curvature has to be subtracted when, as in this case, the observed angle is an angle of depression and added when it is an angle of elevation.

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CHAPTER XVI

LEVELLING

In this chapter the principles of levelling as practised by the civil engineer and surveyor are considered. The principles and use of the mechanic's level and the modern water level are dealt with briefly. The essential elements of construction of the Dumpy level, Wye level and staff in their various forms are explained, together with the methods of using them to find the relative elevations of two or more points. The methods of entering the readings in the level book and computing the levels by the "Rise and Fall" and "Instrument Height" methods are also dealt with.

Levelling.—The operation of levelling has to deal with the determination of the relative heights or differences of elevation of points or objects, usually at some distance from each other. Levelling is used for the two following purposes :—

(*a*) To find the relative elevations of existing points. Such, for example, as finding the elevations of points on the surface of the ground distributed over the site of intended works, information which is usually required to enable the works to be designed.

(*b*) To set out points at predetermined differences of elevation. This is required in the setting out of all kinds of engineering works.

Level Surface.—A level surface is accurately defined as a curved surface which at each point is perpendicular to the direction of gravity at that point. A plumb-line gives the direction of gravity at any place. The surface of still water is a truly level surface. A horizontal line or a horizontal plane at any point is tangent to a level surface at that point. The amount of the downward deflection of a level surface from a horizontal line is given by the formulæ in the preceding chapter, p. 221.

The amount of the deflection at a distance of one-eighth of a mile from the tangent point is $\frac{1}{8}$ in. and increases rapidly for greater

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distances. For most practical purposes, therefore, a horizontal plane at a point may be taken as coinciding with a level surface through the point, over a circular area having a radius of, say, 200 yards.

Levelling Instruments.

Levelling Instruments.—In the levelling instruments used by the surveyor or engineer the action of gravity is employed in various ways to indicate a horizontal line. In the Water Level the surface of still water in two vertical glass tubes, at a distance apart but connected together by a horizontal tube, gives two points at the same level. In the Reflecting Level a small glass mirror is suspended so as to hang with its reflecting surface perfectly vertical. A line from an observer's eye placed at some distance from the mirror to the centre of the image of the pupil seen in the mirror is then a horizontal line. The most common appliance however for indicating a horizontal line and the one which is capable of giving by far the most accurate results is the Spirit Level. In this the indication is given by a bubble of vapour contained in a curved glass tube nearly filled with alcohol or ether and sealed at the ends. Under the action of gravity the bubble rises to the highest part of the tube so that when the bubble has come to rest a tangent to the inner surface of the top of the tube at the centre of the bubble is a horizontal line.

Water Level.—An early form of surveyor's levelling instrument was the water level. This consisted of two short lengths of glass tube connected vertically to the ends of a horizontal metal tube several feet long, the whole being mounted on a tripod. The tubes contained coloured water and, when the instrument was set up and the water allowed to come to rest, a horizontal line of sight was obtained by bringing the eye into line and level with the two water surfaces and looking along them. The instrument was used with a graduated staff and sliding vane.

A modern adaptation of the water level is illustrated in Fig. 191, and consists of a pair of glass tubes 2 ft. or so in length fixed in graduated metal casing frames having broad bases so that they stand erect when placed on the ground. There is a stop-cock and hose connection at the foot of each tube so that they can be joined together by a length of rubber tubing. The water when at rest will

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stand with its surfaces at the same level in the two tubes so that the difference of elevation of the two objects on which the tubes stand will be got by taking the difference of the readings of the water surfaces on the graduated casings. This form of levelling instrument is very useful for working in confined places, such as narrow passages or cellars. The levels in two different cellars, for example, can be found provided there is room to pass the rubber tube from the one to the other. The instrument is also used by mechanics in setting out foundations for machinery, &c.

Spirit Level.—In the "spirit level," or "level tube," or "bubble tube," as it is variously called, the length of the glass tube may vary from about 1 in. in the case of a mechanic's hand level to 7 or 8 ins. in the case of a tube for a sensitive surveyor's level. The

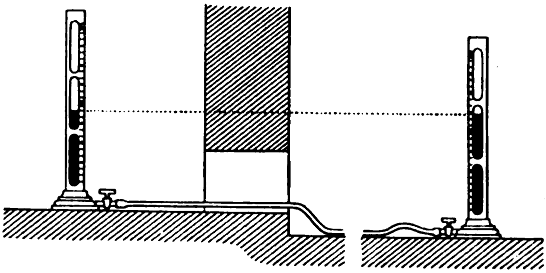

Fig. 191.—Mechanic's Water Level.

smaller bubble tubes consist simply of a piece of plain round glass tube bent to a short radius, and having a single central index mark. The appliance indicates "level" when the small bubble is symmetrical about the index mark. The longer and more sensitive tubes have their top inner surfaces accurately ground to a flat circular curve. The tube is graduated from the centre both ways, and the long bubble indicates "level" when its ends are at symmetrical marks on each side of the centre. The bubble in the tube of a 14-in. surveyor's level will usually have a length of about 3 ins., but the length varies considerably with temperature, being governed chiefly by the expansion and contraction of the liquid. The higher the temperature the greater is the volume of the expanded liquid, and in consequence the smaller is the length of the bubble.

A line tangent to the inner top surface of the tube, at the centre

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of the graduations, which is a horizontal line when the bubble is central, is known as the axis of the bubble tube. This is illustrated in Fig. 192. The essential requirement of a levelling instrument is that the axis of the bubble tube should be parallel to the plane of the supports or parallel to the line of sight of the telescope, according

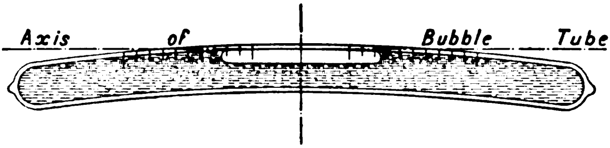

FIG. 192.—Axis of Bubble Tube.

as the instrument is of the form of the mechanic's level or of the surveyor's level.

Mechanic's Level.—A common form of the mechanic's level is illustrated in Fig. 193. The bubble tube is fixed in a hardwood casing which is covered over with a metal plate. The plate is open over the bubble with the exception of a narrow bar across the centre of the tube which serves as an index. There are side slots under the covering plate to enable the bubble to be viewed from the lateral positions. The extremities of the base are protected by metal mountings. In an accurate level the plane of the base must be exactly parallel to the axis of the bubble.

The mechanic's level may be sometimes useful for levelling over short distances in confined places. The instrument is used, in conjunction with a wooden straight-edge generally from 6 to 10 ft. long, as shown in Fig. 194. Assuming that a point has to be established some distance off at the same level as the step at A, one end of the straight-edge will be placed on the step, the level will be laid on the top of the straight-edge at its centre, and the end B will be raised or lowered till the bubble is in the centre of its run, and then packed up from the ground or in some other way temporarily fixed at this level. The top of the support at B will be at the same level as the step at A. By shifting the straight-edge forward with one end resting on B another support may be fixed

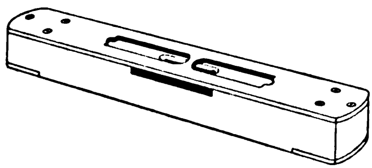

FIG. 193.—Mechanic's Level.

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ahead at the same level as the starting point, and so on till the whole distance is traversed. Errors due to inaccuracy of the level and want of parallelism of the straight-edge will be practically eliminated by reversing the ends of the straight-edge and level at each succeeding length. Thus, if A and a be considered as the rear ends of the straight-edge and level respectively for the first length they should be made the forward ends A′ and a' for the second length, and so on alternately. By this means any error which may occur in an upward direction in one length will be balanced by an equal downward error in the next length.

The mechanic's level is sometimes useful for taking cross-sections on very steep ground where the setting up and manipulation of the surveyor's level is awkward and troublesome. The method is explained on p. 269.

The Level.—The levelling instrument employed by surveyors and engineers for the purpose of determining the relative elevations

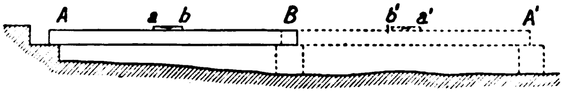

FIG. 194.—Use of Mechanic's Level.

of points at distances apart is generally known simply as the level. Its essential elements are a telescope with a line of sight defined by a horizontal cross hair; a sensitive spirit bubble attached to the telescope and arranged with its axis parallel to the line of sight of the latter; a tripod stand to hold up the telescope to a convenient height for the observer's eye, and an arrangement of levelling screws between the stand and the telescope by which the line of sight of the latter can be made horizontal, as shown by the bubble of the spirit level coming to the centre of its run. The telescope can rotate about a vertical axis so as to command sights in any direction.

The level is commonly constructed in two different forms, which are known respectively as the Dumpy level and the Wye level. A variation of the Dumpy level, as manufactured by Messrs. Troughton and Simms, may also be noted. An elevation of the ordinary form of Dumpy level is shown in Fig. 195, while Figs. 196 and 197 show, for the sake of comparison, corresponding elevations of the Troughton and Simms Dumpy level and of a Wye level respectively.

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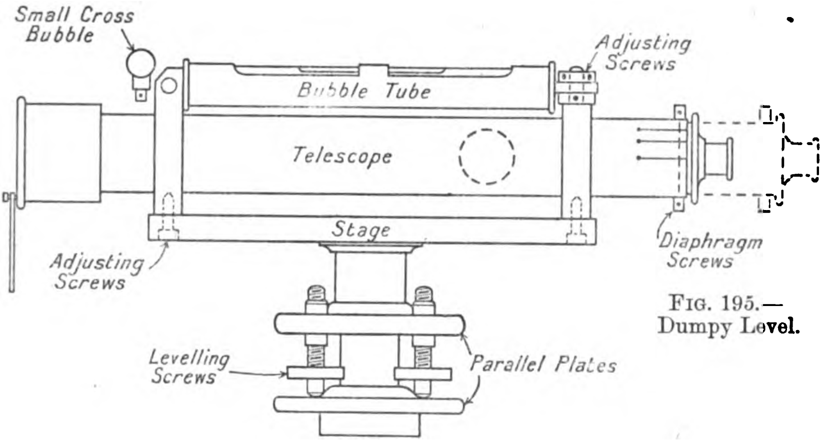

FIG. 195.—Dumpy Level.

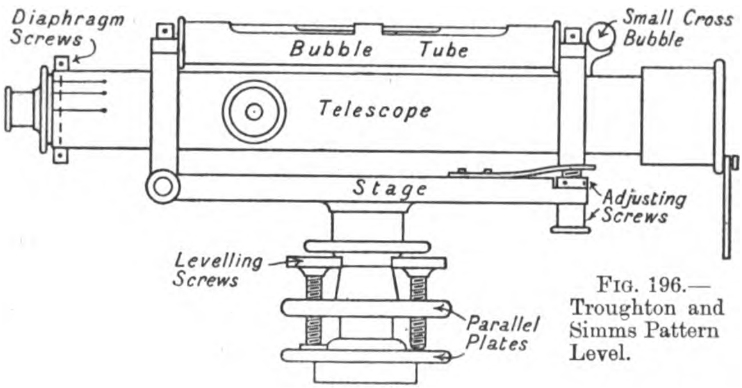

FIG. 196.—Troughton and Simms Pattern Level.

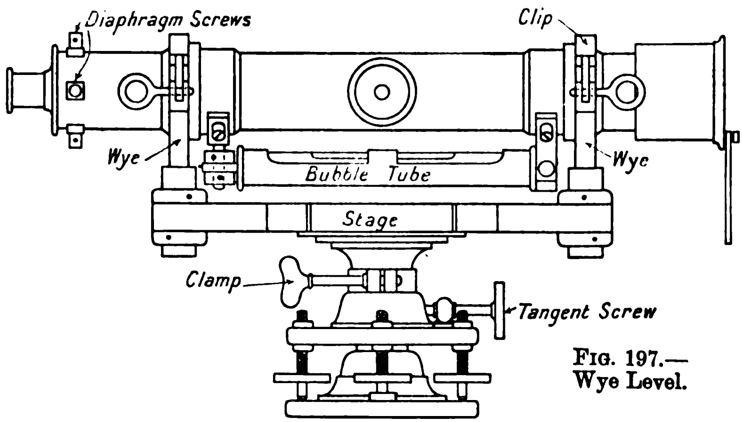

FIG. 197.—Wye Level.

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Dumpy Level.—The telescope of the level is generally similar to that shown in Fig. 129 for the theodolite, but larger and more powerful. Common lengths of telescope are 12, 14, 16, and 18 ins.,

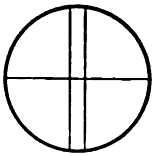

FIG. 198.

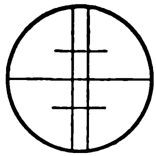

FIG. 199.

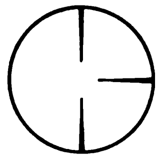

FIG. 200.

the 14-in. size being perhaps the most usual for ordinary work. The eyepiece is fixed in a tube which usually slides within the tube containing the object glass. The movement for focussing the object glass generally takes place from the eyepiece end. Fig. 198 illustrates the form of diaphragm having a single cross hair for giving the reading on the staff and two vertical hairs to indicate whether the staff is held erect. Figs. 199 and 200 show other forms of diaphragm. The diaphragm is adjusted in position within the inner telescope tube so as to have its cross hair horizontal when the instrument is levelled up, and is attached to the top and bottom of the tube by the two opposing capstan screws shown near the eyepiece end, and its sides are held within vertical grooves. The capstan screws permit of the vertical adjustment of the diaphragm. The bubble tube is supported on the top of the telescope, one end of its casing being attached by a hinged joint, while the other end is held between a pair of capstan nuts on a short length of vertical screw attached

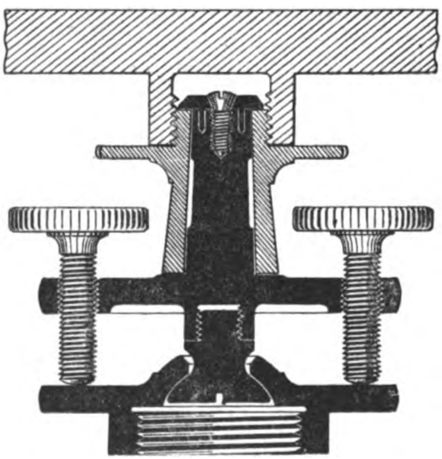

FIG. 201.—Four-Screw Levelling Arrangement.

to the top of the telescope tube. This attachment permits of the end of the tube being raised or lowered slightly, so as to alter the inclination of the bubble axis with respect to the telescope.

The telescope is supported on a horizontal bar or stage formed, preferably, in one piece with a vertical spindle which rotates within a socket attached to the upper parallel plate.

The parallel plates in the four-screw type of levelling arrangement are connected by a ball-and-socket attachment, the arrangement being as shown in Fig. 201.

In the three-screw type of levelling arrangement, shown in Fig. 202, the screws themselves perform the function of the ball-and-socket attachment. The parallel plates are replaced by a pair of tribrach castings. The thumb screws are screwed into the extremities of the upper casting, and their lower ends which have enlargements in the form of a ball or a cone are held by or clamped to the lower casting in such a way as to permit of tilting to a certain extent, as required in the levelling up of the instrument.

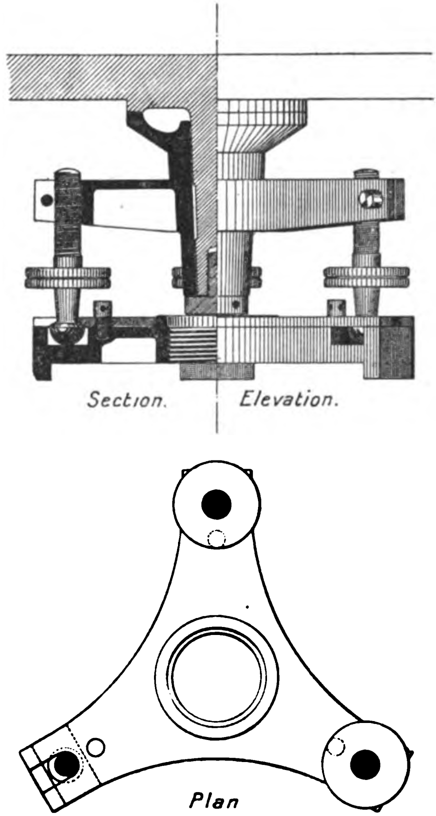

Fig. 202.—Three-Screw Levelling Arrangement.

The tripod stand is most commonly of the solid wooden type, having legs of rounded triangular section which fold together into a tapering cylindrical form. The framed type of stand is superior and for the sake of extra steadiness is sometimes used for the larger instruments, such as the 16-in. and the 18-in.

The following variations of the Dumpy level as above described

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are frequently met with. A small bubble tube is generally fixed on top of the telescope in a direction at right angles to the large bubble. The preliminary levelling-up may then be done in both directions without rotating the telescope, the final precise levelling in each direction being afterwards effected by means of the large bubble alone. A circular bubble is sometimes fixed for the same purpose.

The large bubble is sometimes suspended beneath the telescope. In this position it is not so much exposed to accidental injury as when it is on the top of the telescope.

A small magnetic compass is often fitted on the top of the stage underneath the telescope. . It is only of very limited use for taking bearings, as the telescope of the level cannot be tilted up or down to take sights to points which are much above or below the horizontal plane through the instrument.

It is a convenience to have a clamp and tangent screw on the vertical axis of the larger power instruments for ease in setting the line of sight on to the staff and fixing it there. A clamp and tangent screw are almost a necessity where the diaphragm is of the metal-point form as this requires very exact setting, and are also indispensable if the compass is to be used.

The main requirements of a level in accurate adjustment are the following :—

(*a*) The line of sight of the telescope should be parallel to the bubble axis.

(*b*) The bubble axis should be at right angles to the vertical axis of rotation.

Requirement (*a*) is a necessity to accurate levelling. Requirement (*b*), however, is for convenience to permit of rapid levelling, and accurate results can be obtained although it is not complied with. If (*b*) is fulfilled, then, when the instrument has been set up and levelled, the telescope can be rotated so as to point in any direction and the bubble will always remain in the centre of its run. A series of sights can, therefore, be taken from one position of the instrument without further levelling up. If (*b*) is not fulfilled the bubble requires to be brought to the centre by means of the plate screws for each new direction of sight.

The methods of making the adjustments to bring a level into compliance with the above-mentioned requirements are described

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in Chapter XXII. The differences between the several forms of level, from the point of view of the person who has to use them, lie principally in the provisions made for adjustment.

Troughton and Simms Level.—In the Troughton and Simms pattern of Dumpy level the bubble tube is permanently fixed in position on top of the telescope tube by the instrument maker and is not intended to be altered or interfered with except in case of breakage.

The telescope is attached to the horizontal stage by a hinge at one end and by capstan screws at the other which work against a stiff spring. By turning these capstan screws so as to raise or lower the end of the telescope the bubble axis may be adjusted so as to be perpendicular to the vertical axis of rotation, without affecting the relative disposition of the bubble axis to the line of sight. The provision for adjustment of the diaphragm is the same as in the Dumpy level.

Wye Level.—In the Wye Level, as illustrated in Fig. 197, the telescope is a separate and detachable portion of the instrument. Two circular collars of exactly equal diameter are formed on the exterior of the telescope tube and these rest in the wyes or supports on top of the stage. The wyes are either V-shaped, in which case the collars rest on only two points, or (in some modern instruments) they are circular. A hinged clip passes over the top of each collar to hold the telescope in position and is fastened with a pin or a spring catch. The telescope can be lifted out of the wyes and replaced end for end, and it can also be placed with the bubble tube uppermost as well as underneath, the latter position being the normal one and the only one in which the bubble can be read. These facilities enable the Wye level to be more easily adjusted than the Dumpy. The Y-supports are fastened to the horizontal stage by screw attachments which permit of the tilting of the telescope to a slight extent as required in adjusting the bubble axis to render it perpendicular to the axis of rotation as required in (*b*). The diaphragm of the Wye level is fixed by two pairs of capstan screws so as to be adjustable in both the vertical and horizontal directions.

The Staff.—The simplest form of level staff as regards construction and graduation is the solid wooden pattern which is generally made

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in three lengths with socketed joints giving a total length when put together of from 14 to 16 ft. It is divided into feet and tenths of a foot by black division lines which are each $\frac{1}{100}$ ft. in thickness. This form of staff is illustrated in Fig. 203. The divisions marking the whole foot-lengths are numbered in large figures with the exception of the divisions at 5, 10, and 15 ft., which, to avoid mistakes, are marked by the Roman numerals V, X, and XV respectively. Each intermediate $\frac{1}{2}$ ft. is indicated by a black diamond on the centre of the graduation mark, and sometimes each $\frac{1}{4}$ ft. is indicated by a small black circle. Both sides of the staff are graduated alike. The staff is slightly hollowed on each face, the cross-section being as shown in Fig. 204, with the view to preventing the rubbing off of the graduations.

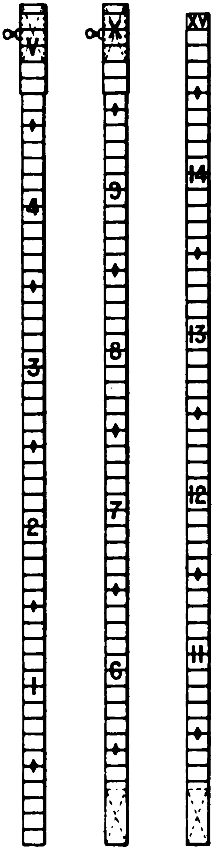

Fig. 203.—Solid or Scotch Staff.

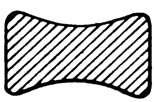

Fig. 204.—Section of Staff.

In reading the staff attention must be very carefully given to the fact that the telescope gives an inverted view. The figures, therefore, appear upside down and the graduations run from the top of the field of view downwards. The operations in reading the staff are :—

(1) Look for the figure which is apparently above the cross hair and note it.

(2) Count the number of whole tenths of a foot down to the division mark immediately above the cross hair.

(3) Estimate by the eye the decimal part of a space from the latter division mark down to the cross hair.

(4) The separate distances obtained in (1), (2) and (3) being added together give the staff reading.

With practice the above operations can be done almost simultaneously and the reading can be obtained practically at a glance. When using the level constantly one becomes scarcely conscious that the staff appears upside down.

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Fig. 205 shows a portion of a staff as seen in the field of view of a telescope. The reading shown is 2·73 (not 3·26). The length of staff which appears in the field of view is proportional to the distance from the level. When the distance is very short the length of staff seen may be less than 1 ft., and may so occur that no figure is visible. In that case the staff-holder is instructed to slowly lift the staff. The observer looking through the telescope at the upper part of the field watches the staff as it appears to move downwards and notes the figure which first emerges. This gives the number of whole

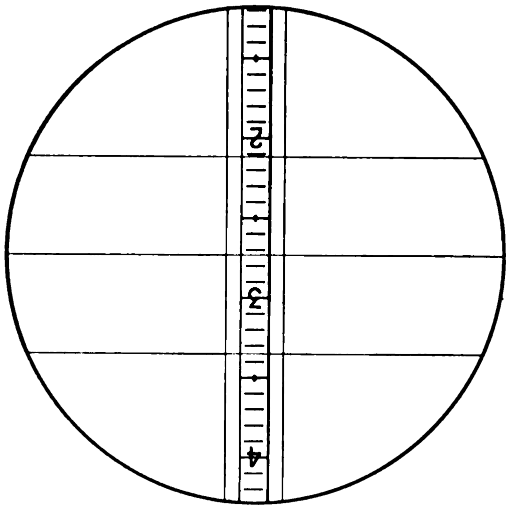

FIG. 205.—Portion of Staff seen in Telescope.

feet, and the staff is then lowered on to the point and the tenths and hundredths are read.

The top of each black division line and not its centre should be taken as indicating the exact graduation. The form of staff above described is frequently graduated as shown in Fig. 206. Each $\frac{1}{10}$ ft. is halved by a short division mark, and the exact positions of the hundredths are then as shown in the figure.

The form of staff known as the "Sopwith" is illustrated in Fig. 207. Usual lengths are from 14 to 18 ft., there being three sections which telescope together for convenience in transport. It is fully graduated to $\frac{1}{100}$ ft., and elaborately numbered and

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marked. The graduations are on one side only. The large figures shown on the left-hand side in the figure mark the whole foot lengths, the top of the figure and top of the horizontal line indicating the exact position of the graduation. The smaller figures 1, 3, V, 7, 9, which are repeated on every foot length on the right-hand side, indicate the odd decimals, the tops of the figures again marking the exact graduation, and, as the figures are exactly $\frac{1}{10}$ ft. in height, their lower edges mark the even decimals.

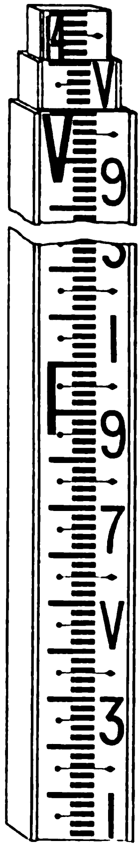

Fig. 207.—Sopwith Staff.

As regards the relative advantages of the two forms it may be said that for very accurate work where large instruments are used and readings are taken to thousandths of a foot it is necessary to use a staff fully and accurately graduated to hundredths. For the great bulk of ordinary levelling, however, the simple staff with open graduations suffices. It is more easily and much more rapidly read, and gives results which in ordinary work are not inferior in accuracy to those obtainable with the Sopwith staff.

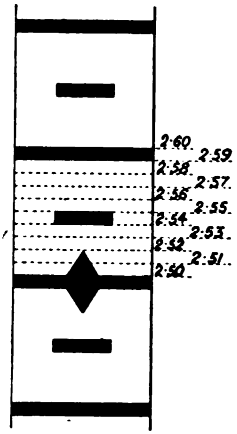

Fig. 206.—Alternative Graduation of Staff.

Field Work.

Use of Level and Staff.—In choosing the position in which to set up the level the endeavour should be made to find a place from which the staff can be read on all the points within working range of the instrument whose levels are required. That is, the point on which the level is planted should, if possible, be at such an elevation that the line of sight of the telescope will not pass above the top of the staff nor below the bottom of it when held on any of the points. As regards the position in plan, subject to the above, this can be any-

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where within range, and should be chosen to avoid obstructions occurring on the line of sight towards any of the points.

The instrument should be set up with a fairly wide spread of the legs to ensure steadiness. Attention should first be paid to the condition of the levelling screws and the focussing of the eyepiece. The plates should be made almost parallel by turning the levelling screws, and the eyepiece should be focussed on the cross hairs in the manner described for the telescope of the theodolite, p. 127.

Before pressing the legs into the ground bring the instrument nearly to the level by moving their points closer in or further out, and by swinging them laterally with reference to the tripod head. This preliminary levelling will be sufficiently accomplished when the whole of the large bubble is visible in any position of the telescope. Then press the legs firmly and evenly into the ground one after the other, and again bring the bubble nearly central by a slight extra pressure on one or two of the legs as required. A little care spent on this preliminary levelling by the tripod legs saves considerable time in the manipulation of levelling screws, and, what is also of importance, saves unnecessary wear and racking of the screws. The final and accurate levelling is undertaken by means of the levelling screws. The telescope is placed in line over a pair of diagonally opposite screws (in the four-screw type), and these are turned simultaneously and evenly in opposite directions till the bubble is brought to the centre of its run. The telescope is then turned through 90°, so as to be in line over the other pair of screws. The bubble will have moved away from the centre, and is brought back by turning this pair of screws. On turning the telescope back into the first position its level condition will be found to have been slightly upset by the movement of the second pair of screws. The operations of levelling-up in two perpendicular directions must, therefore, be repeated a time or two till the bubble is found to remain central in every position of the telescope. In making the first two levellings by means of the screws it is not advisable to spend time in waiting till the bubble comes absolutely to rest and exactly to the centre. Bring the instrument as rapidly as possible, approximately, to the level in both directions and then proceed to exact levelling.

In the case of the three-screw levelling arrangement the telescope is first placed parallel to a pair of screws and then in a perpendicular

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direction over the remaining screw, the levelling of the bubble being accomplished in the former case by turning the two screws in opposite directions, and in the latter by turning the single screw.

To find the difference of elevation of two points, A and B, both within range of the instrument, say, at distances not exceeding 300 ft. from the level. The staff is held vertically on point A. The observer at the level directs his telescope on to the staff, brings it into correct focus by turning the thumb-screw on the right-hand side of the tube, and adjusts till the cross hair appears motionless against the staff as the eye is moved up and down. Before reading the staff, attention should again be paid to the bubble which, if not exactly central, should be made so by turning the pair of screws which are most nearly in line with the telescope. The staff reading is then taken in the manner already described, and booked. The point where the cross hair cuts the staff is at the level of the line of sight of the telescope, and the staff reading, therefore, gives the vertical distance of point A below the horizontal line of sight. When the staff reading at A has been taken and checked by repeating the reading the staff-holder is signalled to proceed to B. The telescope is directed towards and focussed on the staff, *the bubble is brought to the centre if it is found to have moved*, and the staff reading is taken all as before. The staff reading on point B gives the vertical distance of B below the horizontal line of sight of the level. The staff readings on the two points are, therefore, depths measured down from the same datum, and the difference of these depths will give the difference of elevation of the points. Note that the point which has the greater staff reading is at the lower elevation.

Signals.—The observer at the level can tell by comparison with the vertical hairs whether the staff is being held truly vertical or not in the plane at right angles to the line of sight. The usual method of signalling to the staff-holder that the staff requires plumbing is by holding up the arm vertically and slowly inclining it in the direction in which the staff should be moved. The observer must remember that the directions, as seen in the telescope, are reversed so that he must give the signal to tilt the staff further over in the direction in which it seems to be already inclined.

A single wave of the right hand is the customary signal to indicate

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that the staff has been read and that the staff-holder is to proceed to the next point.

Both arms held up vertically indicates that the observer purposes shifting the level to a new point, and that the staff-holder is to take a " change point " as described later.

Datum.—The most convenient method of stating and comparing the elevations of different points is to refer them all to a common level surface, known as a " datum surface," or simply " datum." The elevation of each point will then be expressed as so many units of length, usually feet, above or below the datum. Heights above the datum are positive elevations, depths below the datum are negative elevations. To avoid the inconvenience of figuring and working with negative elevations the datum should always be chosen to come below the lowest point whose elevation has to be referred to so that all elevations will be positive.

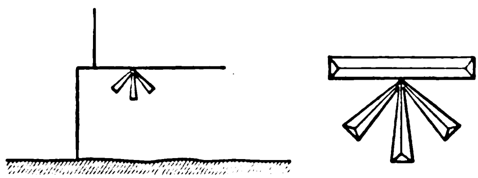

FIG. 208.—Bench Marks.

Bench Mark.—A well-defined and permanent object or mark whose elevation has been determined so as to be available for future use is known as a " bench mark." In connection with the Ordnance Survey of Great Britain bench marks known as Ordnance bench marks have been established all over the country principally along the lines of the highways. They are generally cut near the ground on the masonry of permanent buildings, walls, gate pillars, &c., and consist of a horizontal V-groove with a broad arrow underneath. The centre line of the horizontal V-groove marks the exact position of the determined elevation. A flat plinth is sometimes used as the bench mark with a broad arrow only to indicate the exact position (Fig. 208). The positions of the Ordnance bench marks and their elevations above Ordnance datum are shown on the $\frac{1}{500}$, $\frac{1}{2500}$ and 6 in. to mile Ordnance maps of Britain. The elevations are generally reliable, except in mining districts.

The surveyor in establishing bench marks for his own use will

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usually desire to avoid cutting marks. He therefore makes use of points such as plinths of buildings, ends of doorsteps where unworn, window sills, bases of lamp posts, tops of gate hinges, or any permanent and prominent points which happen convenient to his purpose. The exact positions of such bench marks must be carefully recorded by means of sketches and descriptions.

Ordnance Datum.—All elevations of bench marks, surface of ground, contours, &c., shown on the Ordnance Survey maps are

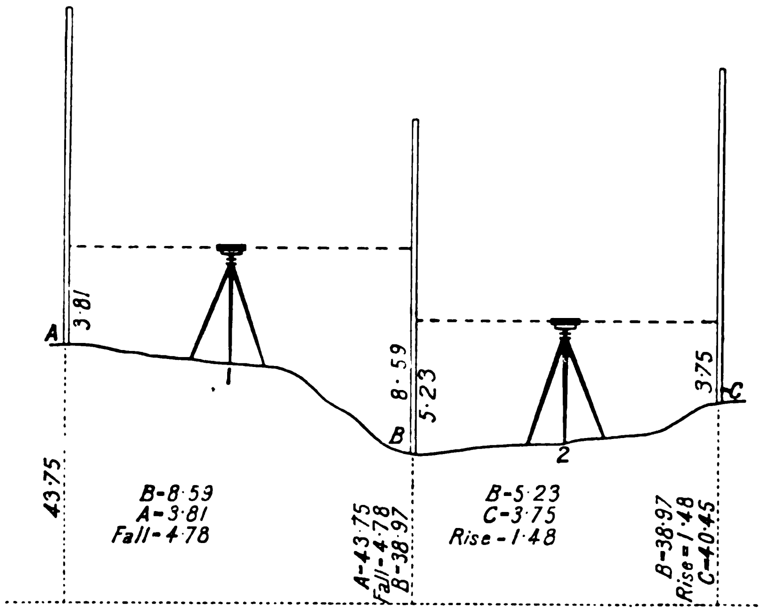

FIG. 209.—Levels by Rise and Fall Method.

referred to the "assumed mean level of the sea at Liverpool," which is commonly known as "Ordnance datum." It is usual in this country to refer the elevations of points, either to Ordnance datum or to some datum at a round number of feet above or below Ordnance datum. In the case of harbour work the datum would be taken at, say, 50 or 100 ft. below Ordnance datum to ensure that all foundation levels, &c., would come above the datum and, therefore, not require to be expressed as negative elevations.

Calculation of Levels.—The calculation of "levels," that is, the heights of points above a datum, may be made from the staff

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readings in two different ways, known respectively as the "rise and fall" method and the "instrument height" method, and illustrated in Figs. 209 and 210. In Fig. 209 the relative elevations of points A and B are fixed by the staff readings of 3·81 ft. and 8·59 ft. respectively taken with the level in position No. 1, and the relative elevations of points B and C are fixed by the staff readings 5·23 and 3·75 taken from position No. 2. The level of point A with respect to a datum is known to be 43·75. The difference of the staff

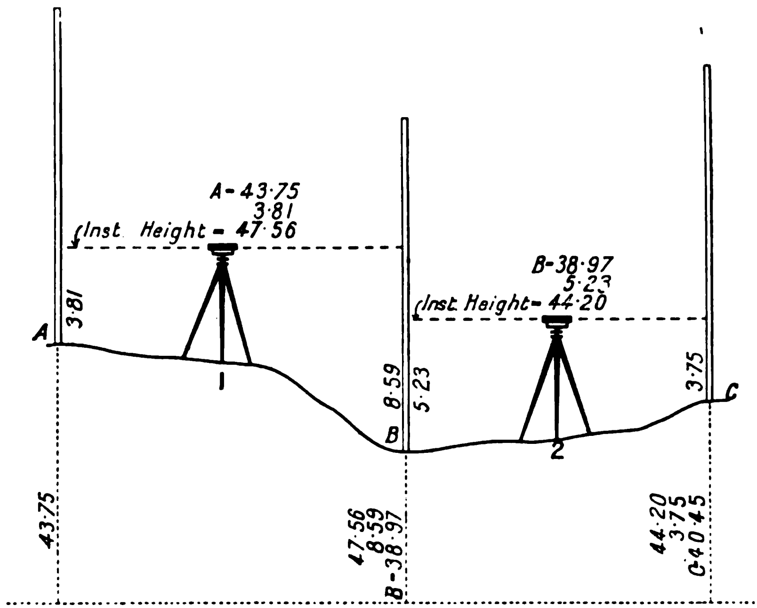

FIG. 210.—Levels by Instrument Height Method.

readings shows that in proceeding from A to B there is a fall of 4·78 ft., and in proceeding from B to C there is a rise of 1·48 ft. The elevation of point B is, therefore, 4·78 ft. less than the elevation of point A and equal to 43·75 — 4·78 or 38·97. The rise from B to C added to the elevation just found for B will give the elevation of C, which is, therefore, equal to 38·97 + 1·48 or 40·45.

In the "instrument height" method, shown in Fig. 210, the staff reading on point A added to the level of point A gives the elevation of the line of sight, or the "instrument height," as we call it, for the readings taken from position No. 1. The level of any other point

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sighted from position No. 1 will be got by subtracting its staff reading from the instrument height. Instrument height for position No. 1 $= 43{\cdot}75 + 3{\cdot}81 = 47{\cdot}56$. Level of point B $= 47{\cdot}56 - 8{\cdot}59 = 38{\cdot}97$. The staff reading on B from position 2 added to the level of B will give the instrument height for position 2, namely, $38{\cdot}97 + 5{\cdot}23 = 44{\cdot}20$. The levels of all other points sighted from position 2 will be got by subtracting their staff readings from the instrument height; thus, level of point C $= 44{\cdot}20 - 3{\cdot}75 = 40{\cdot}45$.

Continuous Levelling.—When two points are at such a distance from each other that they cannot both be within range of the level at the same time, or when their vertical distance apart is greater than the height of the staff, then the difference of elevation of the points cannot be determined by a single setting up of the instrument. In such cases the distance between the points must be divided into stages by intermediate points on which the staff is held, the difference of elevation of each succeeding pair of intermediate points being found by a separate setting up of the level. This process is known as continuous levelling. The term is applicable to any levelling in which a connected series of elevations is obtained from a number of successive positions of the level. If A and B are two points, say, 1,600 ft. apart we may choose three intermediate points, Nos. 1, 2 and 3, dividing the distance into four stages of about 400 ft. each. By setting up the level between points A and No. 1 (but not necessarily in line with them) and reading the staff on each point we may determine the difference of elevation of these two points. Similarly, by three additional plantings of the instrument we may determine the differences of elevation of the pairs of points 1 and 2, 2 and 3, 3 and B. In proceeding over the several stages from A to B we may consider increase of elevation or "rise" as positive, and decrease of elevation or "fall" as negative, and the total difference of elevation of the end points will then be got by summing algebraically the differences of the several stages. In practice, however, the staff readings are usually entered in a "level book" specially arranged to facilitate the calculation of the levels of the various points by the "rise and fall" method or by the "instrument height" method.

The more general case of continuous levelling where the levels of a number of points are obtained at each setting up of the instrument in addition to the points required for the carrying forward

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of the levels is illustrated in Fig. 211. The levelling commences from point A, whose elevation above a datum is already known, and is continued from one position of the instrument to another by means of the readings taken on the points B, C, D, the staff being read on these points in each case from two successive positions of the level. Such points are termed "change points." They may either be points whose elevation is wanted, or points specially chosen for the purpose of continuing the levels. Sights taken to points such as *f*, *g*, *h*, solely for the purpose of finding the elevations of these points and not made use of in continuing the levels, are known as "intermediate sights." "Backsight" and "foresight" are terms used to denote the staff readings taken on the change points and on the commencing and finishing points of a series of levels. The commencing sight taken from the first position of the instrument is a "backsight." Of the two sights taken to each change point the one which is taken from the first position of the instrument is a foresight while that which is taken from the second position of the instrument is a backsight. The last sight of a series of levels is usually reckoned as a foresight. When the instrument is set up in a new position, in the ordinary course the first sight taken will be a backsight to the staff which is being held on a change point, all

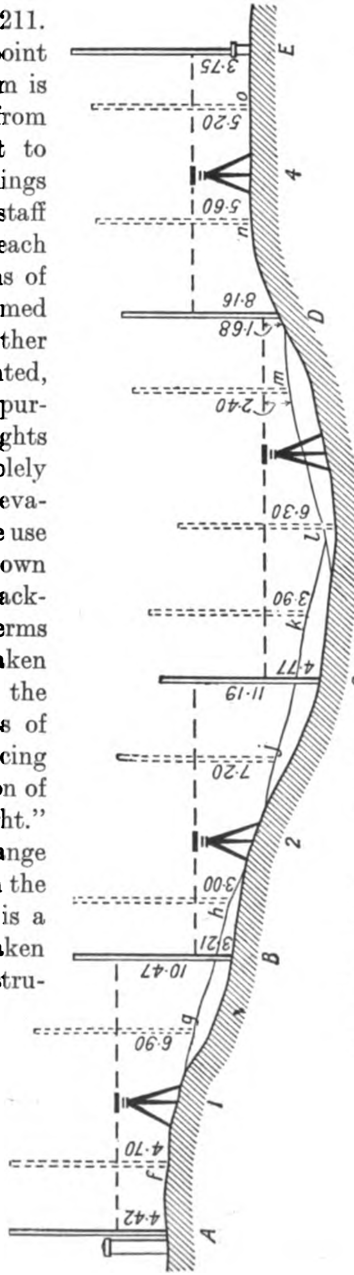

FIG. 211.—Continuous Levelling.

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the intermediate sights will be taken next, and the last sight will be a foresight on to a new change point.

For the sake of simplicity of illustration the positions of the

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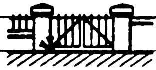
Glenbrook Water Supply
Spot Levels for Conduit

	Back Sight		Intermediate		Fore Sight		Rise		Fall		Reduced Levels		Distance
	4	42									59	87	
			4	70					0	28	59	59	
			6	90					2	20	57	39	
	3	21			10	47			3	57	53	82	
			3	00			0	21			54	03	
			7	20					4	20	49	83	
	4	77			11	19			3	99	45	84	
			3	90			0	87			46	71	
			6	30					2	40	44	31	
			2	40			3	90			48	21	
	8	16			1	68	0	72			48	93	
			5	60			2	56			51	49	
			5	20			0	40			51	89	
					3	75	1	45			53	34	
	20	56			27	09	10	11	16	64	59	87	
					20	56			10	11	53	34	
			Fall		6	53	Fall		6	53	6	53	

Reducing Levels. Rise and Fall Method.

instrument and the positions of the change points have been shown as if they occurred in order alternately along a line. In levelling work generally the places in which the instrument is set up may be disposed in plan in almost any manner with respect to each other, and to the positions of the change points. The terms back-sight and foresight must not, therefore, be taken as in any way implying *direction* of sight.

J. W. / G. B. / W. McL } 20/4/1911
Description
T.B.M. on pivot stone of gate at A
Point f. Field No 128
" g. " " "
" h. " " "
Point j. Field No 131
" k. " " "
" l. " " "
" m. " " "
Point n. on access road
" o. " " "
Check on T.B.M. 53·37 on wall at E.



Reducing Levels. Rise and Fall Method.

Booking and Reducing Levels.

Booking the Readings and Reducing the Levels.—In most forms of level book the left-hand page is arranged for the insertion of the staff readings and the calculation of the levels, while the right-hand

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page is for descriptions of the nature and location of the points and for explanatory remarks. The two pages together are reckoned and numbered as one page.

A form of book appropriate to the "*rise and fall*" method of reducing is shown on pages 244 and 245, with the working out of the levels from the staff readings shown on Fig. 211. Three columns at the left side of the left-hand page are provided for the insertion of backsights, intermediate sights, and foresights respectively. The readings are entered in the order in which they are taken, commencing at the top of the page. The commencing reading, 4·42, taken on point A, is entered in the backsight column. The intermediate sights are then entered on succeeding lines in the column provided for them, and the foresight, 10·47, taken to the change point B, is entered in the foresight column on the next line below the preceding intermediate sight. The next sight taken is a backsight, 3·21, to change point B from position No. 2 of the level, and this is inserted in its appropriate column and on the same horizontal line as the foresight 10·47. The same procedure is followed in entering the set of readings taken from each position of the level.

The further columns on the left-hand page, headed "Rise," "Fall," and "Reduced levels," are used for the working out of the levels. The column headed "Distance" is principally used for recording the positions of points along a chained line in taking longitudinal or cross sections.

The difference between each staff reading and the next one of the same set is calculated and inserted in line with the latter of the two readings in the "rise" or "fall" column, according as the former reading is greater or less than the latter. Thus the difference between the reading on point A and the reading on point *f* is 0·28 and is a fall, that is, point *f* is lower than point A. The number 0·28 is, therefore, written in the "fall" column in line with the reading of point *f*. Similarly, the rise or fall is worked out and entered for each succeeding pair of points. A reference to Fig. 211 will make clear the proper method of procedure at the change points. It is evident that the fall from point *g* to the change point B is the difference between the staff readings on these points taken from position No. 1, that is, it is the difference between the last intermediate sight of the set and the foresight, while the rise from the change point B to the next point *h* is the difference between

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the readings taken to these points from position 2, that is, between the backsight and the next intermediate sight.

The figures for " rise " and " fall " worked out thus for all the points give the vertical distance of each point above or below the preceding one, and if the level of any one point is known the level of the next will be obtained by adding its rise or subtracting its fall, as the case may be. In the example under consideration, the level of point A is known to be 59·87. By subtracting from this the fall of 0·28 to point f we obtain the level of point f. By subtracting the next fall of 2·20 from the level of point f we get the level of point g, and so on.

When the last staff reading (point E) is entered as a foresight the difference between the sum of all the foresights and the sum of all the backsights will be the total rise or fall from the first point to the last point (A to E). The difference between the sum of all the rises and the sum of all the falls entered in the rise and fall columns will also give the total rise or fall from the first point to the last point. A comparison of the results obtained in these two different ways furnishes an efficient check on the calculations of the figures in the rise and fall columns as a discrepancy between the results can only arise from arithmetical error. This check should be applied before the reduced levels are worked out. A complete check on the calculation of the reduced levels is also furnished in a similar way, for the difference between the level of the first point and the level of the last point also gives the total rise or fall between these points. The check is complete, because an error in calculating a reduced level of any point is carried forward as an equal error through all the succeeding points, and, therefore, causes an error in the last point. These arithmetical checks would only fail in the unlikely, but possible, case of two or more errors occurring in such a manner as to balance each other.

A form of level book suitable for the " *instrument height* " method of reducing is shown on p. 248, and, for the sake of comparison, the levels have been worked out for the same set of readings as before. In this level book, instead of the rise and fall columns there is a single column headed " Instrument level," or sometimes " Height of instrument " or " Line of collimation." When the level is in the position No. 1 its line of sight is 4·42 ft. above point A, and the level of the line of sight, or the " instrument height," is, therefore, got by adding the backsight 4·42 to the known level

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59·87 of point A, giving 64·29. This is entered in its proper column in line with the reading of point A. The levels of all the other points, read from the position No. 1, are obtained by subtracting the

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	Inst Height		Back Sight		Inter-mediate		Fore Sight		Reduced Levels		Distance
	64	29	4	42					59	87	
					4	70			59	59	
					6	90			57	39	
	57	03	3	21			10	47	53	82	
					3	00			54	03	
					7	20			49	83	
	50	61	4	77			11	19	45	84	
					3	90			46	71	
					6	30			44	31	
					2	40			48	21	
	57	09	8	16			1	68	48	93	
					5	60			51	49	
					5	20			51	89	
							3	75	53	34	
			20	56			27	09	59	87	
							20	56	53	34	
					Fall		6	53	6	53	

Reducing Levels. Instrument Height Method.

staff readings from this instrument height, the level of the change point B being got by subtracting the foresight 10·47. A new instrument height must now be found for position No. 2, and a reference to Fig. 211 will make it plain that this will be got by adding the

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backsight 3·21 on point B to the level 53·82, just obtained for this point. The rule for the procedure at the change points is, therefore, subtract the foresight from the instrument height to get the reduced level of the change point, then add the backsight to this reduced level to obtain the new instrument height applicable to the next set of intermediate sights and the next foresight.

All the instrument heights and levels of the change points may be worked out and the level of the last point obtained before any of the levels are reduced from the intermediate sights, and this procedure is preferable, as it permits of a check being applied to so much of the calculations. If the difference between the levels of the first and last points is found to be equal to the difference between the sum of the foresights and the sum of the backsights, then the calculation of the instrument heights and change point levels may be accepted as correct and the reduction of the levels of the intermediate points may be proceeded with. It is evident that the calculation for each of the latter points is quite independent and does not affect the calculated level of any other point, so that the instrument height method of reducing does not in itself furnish any check on the accuracy of the calculated levels of the intermediate points. It is, therefore, advisable that the levels worked out by this method should be checked by a second person to ensure their accuracy, and the best way of effecting this is to have the reduced levels worked out independently on a strip of paper, placed so as to cover up the first set of figures, the two sets of results being afterwards compared and any discrepancies investigated.

Another method of checking the levels of intermediate points is as follows: If there are n intermediate levels in the set between backsight and foresight, subtract the sum of the intermediate staff readings from n times the instrument height. The result, if the reducing is correct, will be equal to the sum of the reduced levels of the intermediate points.

Sometimes, instead of having three columns for the staff readings, only two are used, the first for the backsights and the second for the intermediate and foresights, the last reading of each group in the second column being the foresight.

The staff readings may also be booked in a single column, provided some method is adopted for clearly and unmistakably distinguishing between foresights and backsights.

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One method is to draw a line underneath each foresight, that is, beneath each final reading taken on a change point before shifting the level. The following reading placed beneath the line will then be the backsight. Another method is to mark each foresight with an X placed alongside. The accidental omission of any distinction between foresight and backsight would, of course, be a serious blunder, occasioning wrong results in the reduced levels. For this reason the single-column method of booking is unsatisfactory, but where it is used the additional precaution should be adopted of carefully noting the change points on the description page.

The three-column method of booking is the usual one, and is the most generally satisfactory.

Whichever method of booking is used the reduction of the levels may be made by either the " rise and fall " method or the " instrument height " method, in the manners already described.

As regards the comparative advantages of the " rise and fall " and " instrument height " methods of reducing, the latter is the more direct and rapid method as it involves very much less arithmetical work. The " rise and fall " method, on the other hand, possesses the very great advantage of easily furnishing an almost perfect check on the whole of the arithmetical work, whereas the other method provides in itself a perfect arithmetical check for the levels of the change points only. The two methods indicated above for separately checking intermediate levels which have been reduced by the "instrument height" method are somewhat cumbersome. The effect of a mistake in the arithmetic is worth noting in the two cases. In the "rise and fall" method, if one is careful to apply the checks, the occurrence of the mistake will be detected at the foot of the page on which it occurs. The calculations must then be gone over again, starting at the top of the page until the point is found at which the mistake occurs. The reduced levels from this point downwards must then be rubbed out and a fresh set of values calculated and inserted, and the result checked at the foot of the page. In the case of the "instrument height" method an error in the reduced level calculated from an intermediate sight can, as already explained, only be detected by the application of an independent check, but, when found, it merely involves the rubbing out and correction of the particular reduced level involved.

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CHAPTER XVII

ERRORS IN LEVELLING

In this chapter consideration is given to the various errors likely to arise in levelling operations, as enumerated in the following list, and to the relative importance of these errors. The subject is of very great importance and is deserving of most careful study. The surveyor who has a true appreciation of the various circumstances which conduce to the occurrence of error and of the relative importance of the resulting errors will be best able to arrange his work so as to obtain the best results from given expenditure of time and care.

Errors in Levelling.—The principal sources of error in levelling are the following :—

(*a*) Faulty adjustment of the level.
(*b*) Mistakes and carelessness in use of level.
(*c*) Errors resulting from the staff and its manipulation.
(*d*) Inaccuracies and mistakes in reading the staff and mistakes in booking the readings.
(*e*) Curvature and refraction and other natural sources.
(*f*) Mistakes in reducing the levels.

Faulty Adjustment of the Level.—If the level is not in correct adjustment the line of sight will not be horizontal when the bubble is central. The error caused thereby in any staff reading will be proportional to the distance of the staff from the instrument. The error will be equal in amount for all positions which are at equal distances from the instrument, or in other words the line of sight will strike the staff at points of equal elevation if the distances are the same. The effect of imperfect adjustment will, therefore, not be cumulative if the backsight and foresight for each position of the instrument are made practically equal in length, but there will be a certain amount of error in the various intermediate sights if these

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are of unequal lengths. If the backsights happen to be consistently of greater length than the foresights or *vice versâ*, as tends to be the case in levelling continuously uphill or downhill, the error due to imperfect adjustment will be cumulative.

An annoying source of error occurs when the inner tube of the telescope does not move exactly parallel to the outer tube, as shown by the cross hair appearing to move up or down the staff as the thumb-screw is turned for focussing the object glass. This causes an error in the staff readings which varies with the length of sight. Error will be largely eliminated if the sights can be arranged of nearly equal length so that one setting of the focus suffices for each position of the level.

Mistakes and Carelessness in Use of Level.—The bubble may not be brought exactly to the centre of its run in levelling up the instrument. This may happen if the bubble is a sluggish one and is read before it has come quite to rest, or if it is viewed from a slanting direction and not from a position exactly square to the direction of the telescope. For important sights the bubble should be carefully examined after the staff is read as well as before it.

If the eyepiece is not correctly focussed on the cross hairs an error of variable amount may result in the staff readings. The value of the reading will depend on the position of the eye at the moment.

Error may also arise due to the shifting or settlement of the level if it has not been planted firmly enough in the ground, or if it should be accidentally touched or jarred, or if the observer should tread too near the legs.

The lifting of the level before the foresight has been taken is a serious blunder.

Errors Resulting from the Staff and its Manipulation.—The error of graduation of a staff should be almost negligible, but it is nevertheless advisable to test the staff occasionally with a steel tape. Alteration of length may occur due to wear at the joints, or, in the socketed pattern, due to the lengths not being driven home, or due to dirt getting into the sockets. A small error in the length of the staff will cause an accumulative error in the results if the levelling is continuously uphill or downhill. An error of $\frac{1}{8}$ in. in 10 ft. would cause an error of fully 6 ins. in levelling through a vertical height of 500 ft.

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The error due to alteration in length of a well-seasoned wooden staff with change of temperature is for most practical purposes negligible.

Error will be caused if a layer of dirt is allowed at times to adhere to the bottom of the staff, but a layer of constant thickness permanently adhering to the bottom would cause no error.

The principal source of error in connection with the use of the staff arises from its not being held truly vertical. In Fig. 212 the correct distance of the point C below the horizontal line of sight of the level is the length CA read off on the staff held vertically on point C. For an inclined position, such as CB, the reading obtained will be too great by an amount represented by the length BA′. The amount of the error will evidently become increasingly great

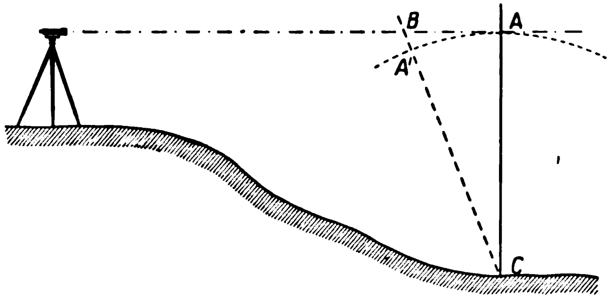

Fig. 212.—Staff not held Vertical.

as the inclination of the staff to the vertical becomes greater, while for a given inclination of staff the error will be in direct proportion to the height of the reading on the staff.

The correct reading of the staff will be obtained by waving it on both sides of the vertical about point C towards and from the level, and noting the lowest reading. This precaution should be adopted for all important points and especially for change points.

It is not desirable, however, to sway the staff for readings near the ground, say, within the lower 2 ft. of the staff, as the graduated face may be raised as shown in Fig. 213, thereby giving a reading A′B′ which is smaller than the reading AB obtained when the staff is vertical. This effect occurs most pronouncedly with a staff having a comparatively broad base, such as the Sopwith.

Error due to inclination of the staff is practically always in the

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direction of making the reading too great. The general effect will, therefore, be the same as if the readings were taken with a shrunk staff. In proceeding continuously uphill or downhill, the backsights will be constantly greater than the foresights or *vice versâ*, so that if any error due to inclination occurs it will tend to be cumulative. In a stretch of levelling where there is no great change of altitude errors due to inclination will tend to be compensative.

Slight settlement of the staff downwards occurring at the change points between the reading of the foresight and the backsight will cause an error equal to the amount of the settlement. The error will be carried forward, and recurring errors due to this cause will be in the same direction and will be cumulative. The staff should always be held on firm and definite change points and should preferably never be removed between the reading of the foresight and the backsight.

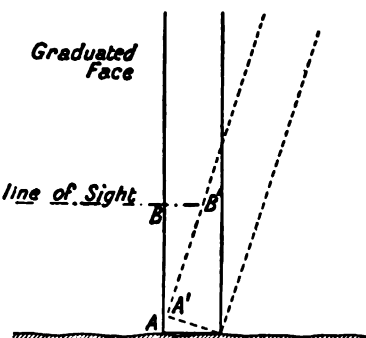

FIG. 213.—Error at Bottom of Staff.

Inaccuracies and Mistakes in Reading the Staff and Mistakes in Booking the Readings.—Referring to Fig. 205, which shows a portion of a staff as seen inverted in the telescope of a level, the correct reading of the centre hair is 2·73. A common mistake with beginners is to read upwards instead of downwards, thus obtaining the wrong reading 3·26. The mistake may also readily be made of reading the decimal portion correctly but taking the wrong whole numeral. For example, in taking the reading shown in the figure, the numeral 3 being nearest the centre cross hair is most prominent to the eye and is apt to be set down instead of the correct numeral 2. The incorrect reading obtained would thus be 3·73.

If the telescope is furnished with stadia hairs, the mistake may be made of reading the wrong cross hair. The upper hair (as seen in the telescope) is the one which is most liable to be read, and in the case illustrated the wrong reading 2·11 would result.

The observer requires to be alive to a fruitful source of error when

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the cross hair occurs in the first tenth of any foot-length. The error consists in estimating the decimal fraction of the first tenth and setting it down in the book as the decimal fraction of a whole foot. Thus if the true reading were 2·04 ft. it might be booked incorrectly as 2·40 ft.

The longer the sight the greater is the apparent thickness of the cross hair compared with the graduations of the staff so that at long sights the true reading may be indefinite to a few hundredths of a foot, and a small amount of error may result in consequence. Long sights should not be taken to any points whose level requires to be accurately determined.

Mistakes in reading the staff can only be avoided by the exercise of constant care. The likely sources of error as noted above should be kept in mind with a view to their avoidance, and in the case of important sights the reading should be repeated once or twice

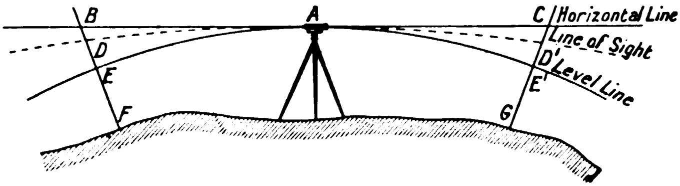

FIG. 214.—Effect of Curvature and Refraction.

after it has been booked, the eye being removed from the telescope between the separate readings.

A mistake in booking which is liable to happen with beginners is the setting down of the foresight in the backsight column or *vice versâ*. The mistake of omitting to book a reading altogether may occur if the observer is spoken to or disturbed after he has read the staff but before he has entered the result in the notebook. The mistake is sometimes made of unconsciously recording numbers with a pair of figures interchanged—for example, writing down 7·53 when the figure in the mind is 7·35.

The staffholder may hold the staff on a wrong point, or may take a group of points in wrong rotation. The result in each case will be equivalent to an error of booking.

Curvature, Refraction, and other Natural Sources.—Fig. 214 illustrates the effects of curvature and refraction. The actual line of

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sight is represented by the dotted line DAD′ which curves slightly downwards from a true horizontal line BAC. The curved line EAE′ represents a level line parallel to the surface of the earth.

The deflection of the actual line of sight below the horizontal at any point is about one-seventh of the deflection of a level line at the same point, but the effect of refraction varies slightly with the state of the atmosphere. The combined effect of curvature and refraction is expressed sufficiently closely by the following formula :—

$$C = \frac{F^2}{48,000,000}$$

where F is the length of sight in feet, and C is also given in feet. Values of the correction C for various distances F are given below.

F in feet =	200	400	600	800	1,000	2,000	3,000	4,000
C in feet =	·001	·003	·007	·013	·021	·083	·187	·333

From the above table it is seen that the error due to neglecting the effect of curvature and refraction is very small for all ordinary lengths of sight. For points at equal distances from the level, such as F and G in the diagram, the errors will be the same in amount, so that if equal backsights and foresights are taken no accumulation of error will result. If the foresights are taken consistently longer than the backsights or *vice versâ*, as may happen in levelling continuously downhill or uphill, the error due to curvature and refraction will be cumulative. If, for example, all the foresights are taken 100 ft. long and all the backsights are taken 400 ft. long the resulting error from this cause will amount to ·02 or ·03 ft. in a mile of levelling.

If the sights are limited to moderate lengths, and the backsight and foresight for each setting of the level are made approximately of equal length, as judged by the eye or measured by pacing, the error due to curvature and refraction will, for most practical purposes, be negligible.

Other natural sources which may give rise to error in levelling are high wind, sun producing atmosphere of variable temperature, sun causing variable heating of the level, frost, and thaw. High wind may cause shaking of the instrument to such an extent as to render accurate levelling impossible. The remedy is either to shelter the level, or wait till the wind falls.

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When the ground is being heated by a strong sun, and causing in turn the warming of the adjacent layers of the atmosphere, the latter may attain a condition of very variable temperature, causing irregular refraction of rays of light. Objects viewed through such an atmosphere appear distorted and of changing form. The graduations of the staff appear to be contracting, expanding, and wriggling in a confusing manner so that readings can only be approximately guessed at and accurate levelling is out of the question. If the work is important the levelling should be delayed till the atmosphere has attained a more uniform temperature.

Variable heating of the telescope and head of the level may cause temporary warping and consequent faulty adjustment. The errors resulting from this may tend to be cumulative if the sides of the instrument are alternately exposed to the sun in taking backsights and foresights. For important work the errors from this source may be avoided by keeping the level carefully shaded from the sun's rays.

On frozen or ice-covered ground the iron-shod points of the legs are liable to a slight, gradual settlement under the weight of the instrument. The conditions are worst when a thaw has commenced, and the amount of settlement of the level and of the staff may then be serious. Errors caused by settlement of the level and the staff are always in the same direction and are cumulative. When levelling requires to be carried on under the above-mentioned conditions the backsight and foresight for each position of the instrument should be taken as rapidly as possible after each other, so that the level gets little time to settle, and the change points should, where possible, be firm, permanent objects which are not ice-covered.

Mistakes in Reducing the Levels.—In the work of computing the reduced levels, involving as it does a large amount of subtraction of figures in different columns, it is hardly surprising if arithmetical mistakes should occur pretty frequently. The permitting of these mistakes to pass undetected and to remain as errors in the work can, however, only be attributed to gross carelessness. Such errors may be almost entirely eliminated by the application of the appropriate checks, as described in Chapter XVI., pp. 247 and 249.

Appropriate Length of Sight.—The distance at which a clear and

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precise reading of the staff can be made depends on the power of the telescope, the nature and distinctness of the graduations and numbering on the staff, and the condition of the light. In sunlight much longer sights can be taken when the view is towards the sunny side of the staff than when it is towards the shady side. It is seldom that precise readings can be made at distances much greater than 600 ft. Long sights may be taken in rapid preliminary work, such as spot levelling, where speed is of importance and great accuracy is not required; also to unimportant isolated points where the reading of a fairly long sight might save an additional setting of the level.

Beginners are apt to have a leaning towards making the sights as long as possible, from an impression that the accuracy and rapidity of the work will be enhanced owing to the reduced number of changes required. From the foregoing discussion of the errors in levelling, however, it will have been seen that taking all sources together the error in a sight increases more rapidly with the length of the sight, so that the error in a single sight of 300 ft. will under similar conditions be less than half the error in a sight of 600 ft. A further reason for the employment of short sights lies in the fact that in practice the total error in levelling over a given distance does not mount up in direct proportion to the number of sights but more nearly in proportion to the square root of the number of sights.

It is probable that for best accuracy the lengths of foresights and backsights should average about three chains or, say, 200 ft., and should not exceed 300 ft. or be much less than 100 ft. There is the further advantage in the adoption of short sights, that the observer at the level thereby has his assistants always within convenient distance for communication, observation, and control, and the work proceeds easily and smoothly and with the least chance of mistakes.

Permissible Error in Levelling.—The degree of accuracy desirable in levelling work depends, among other things, on the nature of the work and the time and money available for its execution. At the one extreme we may place the rough preliminary work required for the location of a road or railway, where the endeavour is to obtain as rapidly as possible a broad determination of the features

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of the country sufficient to locate a route. At the other extreme may be placed the very accurate work required in the setting out of works of the nature of drainage or water supply tunnels, where regular gradients may have to be laid out with a total fall of less than one foot per mile, or the precise work of the Ordnance Survey Department in levelling for the establishment of bench marks, &c., in this country.

The error which accumulates in a stretch of levelling can only be exactly determined when a circuit is completed returning to the starting point. The total accumulated error is then the difference between the elevation assumed for the starting point at the commencement of the levelling and the elevation of the same point found by calculation as the result of the levelling. As already indicated, the error in levelling may be expected to accumulate, not in direct proportion to the number of changes, but more nearly as the square root of the number of changes. The number of changes per mile being fairly constant we may express the error by the formula

$$E = C\sqrt{D}$$

where E is the accumulated error in feet, D is the distance levelled in miles, and C is a constant for levelling of any particular degree of accuracy. The value of C may be taken as a measure of the precision of the levelling.

With ordinary instruments and without special precautions an error of 1 in. or ·08 ft. in a mile would represent fair accuracy. The error in four miles under the same conditions might be expected to be 2 ins. The formula expressing this degree of fair accuracy would be $E = \cdot 08\sqrt{D}$.

Good ordinary work would be represented by the formula $E = \cdot 05\sqrt{D}$, while $E = \cdot 02\sqrt{D}$ would represent very precise work, such as could only be attained by the use of a special instrument and the adoption of special precautions.

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CHAPTER XVIII

SECTIONS, CONTOURS, ETC.

THIS chapter deals with levelling applied to particular purposes such as the taking of longitudinal and cross sections and the obtaining and locating of contour lines. Consideration is also given to methods and devices which may be used in particular cases, such as levelling up a steep slope, levelling over summits and hollows, taking levels of overhead points, levelling past obstructions of various kinds, levelling by the reciprocal method.

Longitudinal and Cross Sections.—In taking a longitudinal section along a line marked out on the ground, the elevations of a series of points on the line are determined by levelling, the positions of the points being simultaneously located by chaining along the line and noting their distances from the point of commencement. Where the vertical profile of the ground is regular or gradually curving levels will be taken on points at equal distances apart and generally at intervals of a chain length. On irregular ground, that is, where abrupt changes of slope occur, the points should be chosen in the positions best suited to accurately determine the section. This will be accomplished by taking the levels of all points where the slope of the ground changes, in addition to the levels at each complete chain length.

In the designing of works which occupy only a narrow strip of ground, such as sewers, water conduits, &c., a longitudinal section along the centre line of the track gives all the information that is required as to the surface of the ground. In the case of works which occupy a strip of ground of some width, such as railways, roads, &c., a longitudinal section along the centre line will serve the purpose if the ground is level across in the direction at right angles to the centre line. But if the ground has a variable cross slope, the information given by the longitudinal section will not be

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sufficient and must be supplemented by means of cross-sections. These are short sections taken at intervals at right angles to the chain line and extending usually a little way beyond the limits of the intended works on either side. In the case of works which are to occupy a broad area of ground the required information as to the features of the surface will usually be obtained by a series of parallel cross-sections taken at right angles to a base line or by several groups of such cross-sections taken from different base lines, and arranged to cover the area in the manner best suited to the circumstances.

Longitudinal Sections.

To run a longitudinal section expeditiously a party of four persons is required, namely, a leader of the party who will work the instrument, book the readings, and direct the operations, a staffholder, and two chainmen. By making one of the chainmen act as staffholder the party may be reduced to three persons, while at a pinch two men can do the work, in which case the operations of chaining and levelling are carried on alternately.

As a rule the levelling will be arranged to start from a bench mark or from some point of known elevation, and will proceed by alternate backsights and foresights till the level is set up in a suitable position to command the commencement of the longitudinal section. Chaining will then proceed and the staffholder will hold on the points at distances, 0, 100, 200, 300, &c., and on such other intermediate points as are required to give an accurate section, or as directed by the chief of the party. The points will be given in the order in which they occur along the section, and the staffholder will call out their distances to the observer at the level, who will book them along with the staff readings. When the limiting length of sight is reached, or when it becomes necessary to shift the level owing to other circumstances, such as the line of sight passing entirely above or below the staff, or the view becoming obstructed, a change point will be taken which may either be a point on the section, if firm enough, or some other point specially chosen, and the foresight will be read. The precautions of swaying the staff, and checking the reading after it is booked, should be observed. The level may then be lifted, carried forward and planted in a suitable position from which to overtake the next portion of the section.

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The backsight will be read as carefully as the foresight and the chaining and levelling of the section will then proceed as before.

For purposes of checking and future reference, temporary bench marks should be established at intervals along the section, say, three or four to each mile. Should any bench mark or point of known elevation occur near the section line the opportunity should be taken of checking on to it, and working out its reduced level from the booked figures there and then. If this does not agree with its previously known elevation, a line of check levels should be run backwards to the point of commencement, picking up on the way any temporary bench marks which may have been established. When the reduced levels of these bench marks and of the commencing point have been calculated from the check levels a comparison with their first determined elevations will show whether any mistake has occurred, and where. If there is a mistake, it will be located as occurring somewhere between two of the bench marks, and the portion of the section affected can then be levelled over again.

Where there are no existing bench marks and only one levelling party is working, the field work will be checked by the method of levelling back to the starting point. A long section would be taken in portions, each portion being checked before the next is levelled, so as to minimise the amount of wasted labour in case a mistake should occur. It is a good plan to arrange the operations so that each day's work stands completely checked by itself.

If speed is important two levelling parties may be employed, one running the section and the other checking, or one party may go forward establishing bench marks, which will be used as checks by the second party running the section, or two parties engaged on different work in the same vicinity may agree to meet at certain pre-arranged places and check each other by levelling on to the same point.

Example of a Section for a Small Sewer.—Plate III. shows a longitudinal section for a small branch sewer. The portion of the level book referring to this section is given on pp. 264 and 265. The levelling is commenced from an Ordnance bench mark, and finishes for purposes of checking on a temporary bench mark which had previously been established in connection with another portion

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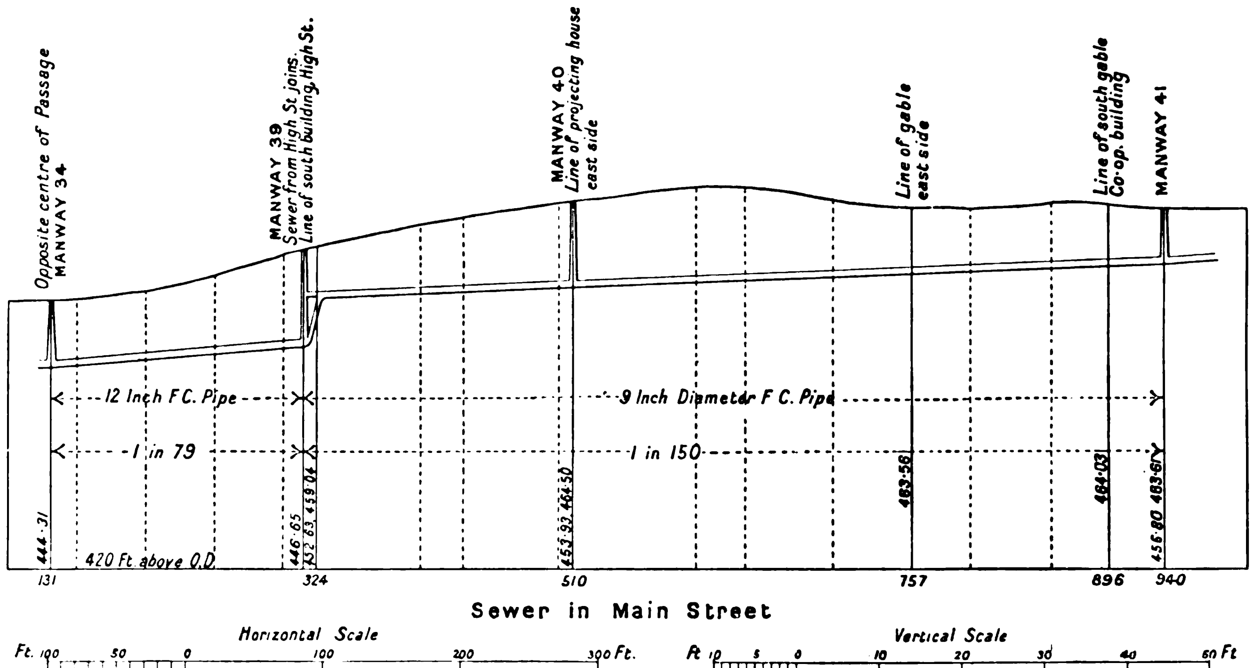
PLATE III. Example of Section for Small Sewer.



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of the same work. In chaining the section distances have been noted to a few easily recognisable points, such as cuts of side streets, lines of prominent buildings, &c., so that the position of the sewer may be accurately laid down on the Ordnance Survey map, which was utilised in preparing the plan of the sewers.

In plotting a longitudinal section the vertical heights will usually be drawn to a much larger scale than the horizontal distances, unless the natural slopes of the ground are steep. In the section illustrated the horizontal scale employed was 1 in. = 60 ft., and the vertical scale 1 in. = 10 ft., giving a ratio of exaggeration of six. The horizontal distances of all points at which levels have been taken are first marked off along a datum line chosen to represent some convenient elevation in round figures, and such as will give a convenient height of section when plotted, having regard to the amount and nature of the information requiring to be marked thereon. Perpendicular lines are then drawn in pencil through these points and the vertical heights above the datum line are marked off to scale giving a series of points on the profile of the ground. A continuous line drawn through these points gives a representation of the surface of the ground to an exaggerated vertical scale.

If a complete record of the section is desired all the vertical lines used in plotting may be inked in, and the corresponding elevations of the surface written in ink alongside. In the section illustrated only such information has been recorded as might be required in the setting out of the works, or in taking out quantities, or for future reference.

In finishing a working section such as this it is customary to show in red the intended new works and all notes and information referring to them, the rest of the section being shown in black.

Example of a Longitudinal Section for a Railway.—Plate IV. shows a portion of a working section for a double line railway, and Plate V. shows the corresponding portion of the working plan.

In connection with the promotion of a parliamentary Bill for a railway in Britain it is a requirement of the standing orders that the distances on the plans and sections shall be given in miles, furlongs and chains (of 66 ft.). The same system of measurement is usually (but not necessarily) adhered to in preparing the working

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plans and sections, and is the system adopted for the section illustrated. The vertical heights are in feet.

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Sewer in Main Street

	Inst. Height		Back Sight		Inter-mediate		Fore Sight		Reduced Levels		Distance
	463	94	1	84					462	10	
					11	40			452	54	100
											131
					11	35			452	59	150
					10	39			453	55	200
					8	48			455	46	250
					5	87			458	07	300
					4	90			459	04	324
											334
					2	19			461	75	400
					1	50			462	44	431
	469	38	5	45			0	01	463	93	
					5	20			464	18	500
					4	88			464	50	510
					3	37			466	01	600
					3	30			466	08	636
					5	02			464	36	700
					5	82			463	56	757
					5	73			463	65	800
					5	05			464	33	858
											896
					5	35			464	03	900
					5	77			463	61	940
							4	67	464	71	

For this section the centre line of the railway was set out with the theodolite, and pegs were driven at each chain from the com-

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mencement. The levelling of the section followed the staking out, the staff being held on each peg and where necessary on intermediate

(Section going South)

Description
O.B.M. on Church 462·1
Line of centre of passage
S. Line of High Street
Centre of Broadlie Road
Turn. Line of projecting house E. side
S. Gable. House nearest road on W. side
Line of front garden wall E. side
Opp. No 124
S. Gable Co-op. Buildings
Turn opp. S. gate-post last House E. side
T.B.M. On stone at corner of railing
last house 464·68

points also. The levels of the latter are made use of in plotting the surface line but are not recorded on the section.

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The horizontal scale of the section is the same as the scale of the plan, namely 1/2500 or 1 in. = 208·33 ft.; the vertical scale is 1 in. = 30 ft. The datum is Ordnance datum, that is, mean sea level at Liverpool.

Of the two parallel lines drawn in the upper portion of the section the lower one represents the formation level of the railway, that is, the finished surface of the earthwork on which the ballast is to be placed. The upper line represents the level of the top surface of the rails. Formation level is generally shown by a red line and rail level by a blue line.

The information to be recorded on the section comprises the following :—

(*a*) The names of important objects or features crossed by the railway, such as roads, streams, railways, &c. These are printed above the section in line with the objects to which they refer.

(*b*) The ground surface level at each peg. These levels are written in black vertically along the lines to which they refer, and form the bottom row of figures on the section.

(*c*) The formation level at each peg. When the gradients have been fixed and the formation line has been drawn on the section the formation level at each peg is calculated, working from the levels which have been established at the changes of gradient. The formation levels are usually written in red ink immediately above the ground surface levels. A thick vertical red line marks the position of each change of gradient, and the formation levels at the changes are printed in bold figures so as to stand out conspicuously from the others.

(*d*) The depth of cutting or height of embankment at each peg. These depths and heights are found by taking the difference between the formation level and the ground surface level at each peg. The result will be cutting or embankment according as the formation level is smaller or larger than the ground surface level. The calculated depths and heights should agree throughout with the corresponding dimensions scaled from the section.

(*e*) The gradients of the railway. These are printed boldly in red above the row of formation levels.

(*f*) The distances along the section. These are printed underneath the datum line. In the section illustrated the distances are marked in figures at ten-chain intervals (660 ft.) giving miles and

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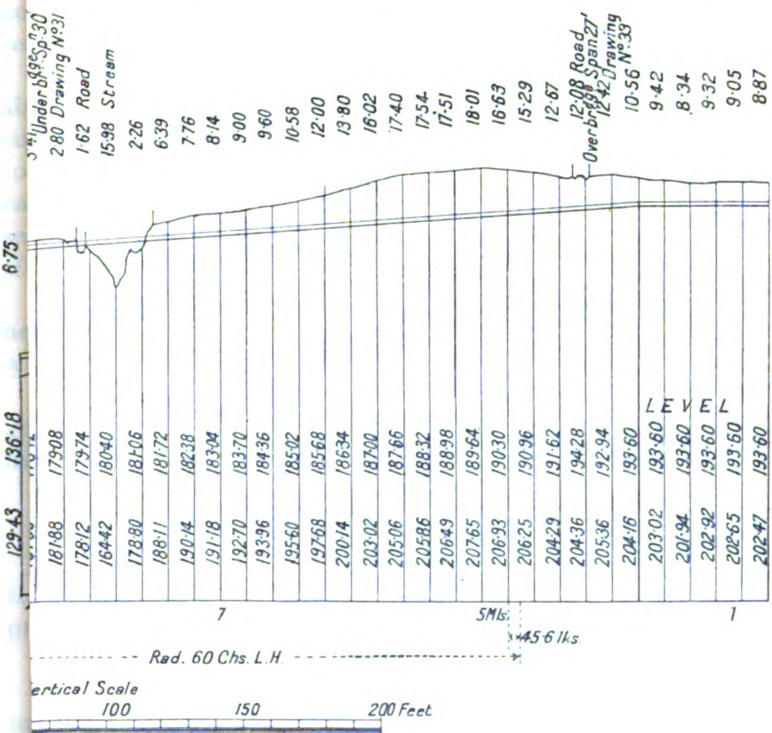
PLATE IV. Portion of Working Longitudinal Section of Railway.



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furlongs from the commencement of the line. Where the pegs have been placed at 100 ft. intervals they are usually numbered consecutively, and instead of distances, the number of each tenth peg from the commencement is printed on the section. Peg No. 237 would thus be at a distance of 23,700 ft. from the starting point of the section.

(*g*) A note of the datum used, a descriptive title of the section, and the vertical and horizontal scales to which the work is plotted, accurately drawn once or oftener, according to the length of the section.

In addition to the above information a railway section should also show the position of all separate items of work, such as bridges, culverts, &c., with a note of their leading dimensions and a reference to the numbers of the drawings on which they are detailed. In the case of a bridge carrying the railway over a road the note on the section would be similar to the following: "Underbridge, Span 40 ft., Headway 16 ft., Drawing No. 37."

The positions and results of any borings or trial pits which have been made to determine the nature of the materials in the cuttings will also usually be shown on the section.

For general information and as a guide in the arranging of the gradients the positions and radii of the curves of the railway may be noted underneath the section as in the example illustrated.

Cross-Sections.

Cross-sections are made use of in connection with the construction of railways, roads, canals and works of like nature for two principal purposes, namely :—

(*a*) To determine the area of ground covered by the works.

(*b*) To determine the quantities of excavation, embankment, &c., in the earthworks.

For rough purposes short cross-sections may be taken with an inclinometer or Abney level. If the ground has a uniform slope at right angles to the longitudinal section, the information requiring to be noted in the level book is the angle of the slope and the direction of the fall, whether towards the right hand or towards the left hand. Right and left hand in connection with cross-sections have reference to an observer standing on the longitudinal section and looking forward in the direction in which it is being run. It

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is important in sketching and plotting cross-sections to see that the slope is never laid off to the wrong hand. Inclinometer cross-

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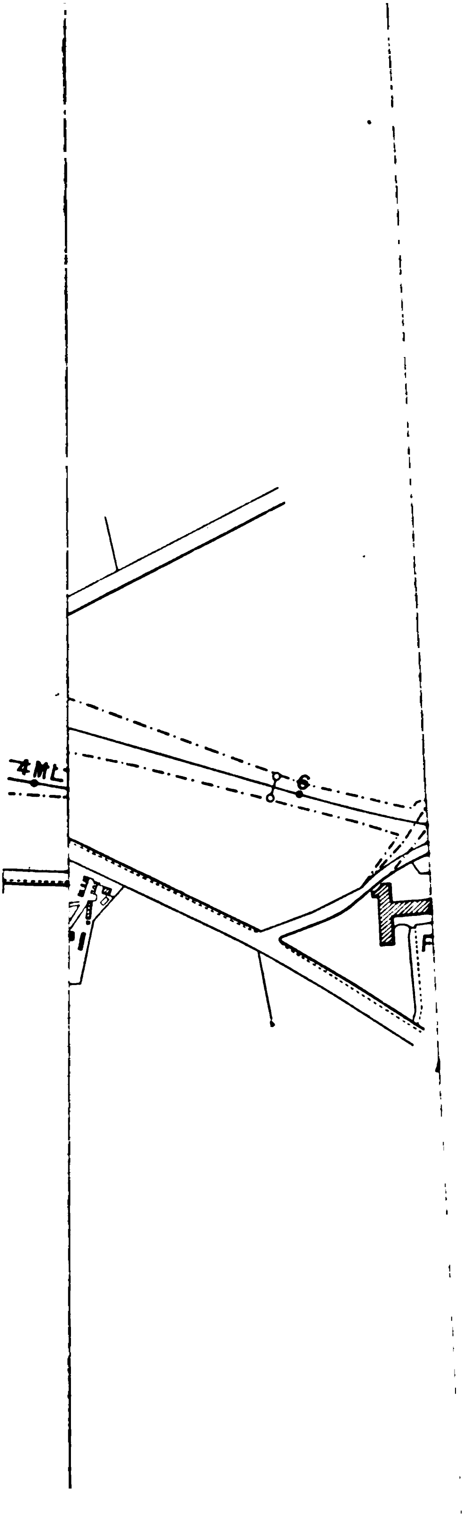
Cross Sections for Railway No. 3

	Inst. Height		Back Sight		Inter-mediate		Fore Sight		Reduced Levels		Distance
	268	80	5	10					263	70	
					6	79			262	01	$8^{\text{m}}\ 14^{\text{s}}$
					9	30			259	50	$8^{\text{m}}\ 15^{\text{s}}$
					11	60			257	20	$8^{\text{m}}\ 16^{\text{s}}$
	258	78	3	50			13	52	255	28	$8^{\text{m}}\ 17^{\text{s}}$
					5	17			253	61	$8^{\text{m}}\ 18^{\text{s}}$
					7	28			251	50	$8^{\text{m}}\ 19^{\text{s}}$
					9	88			248	90	$8^{\text{m}}\ 20^{\text{s}}$

sections may be booked by sketching in a manner similar to that shown on p. 269.



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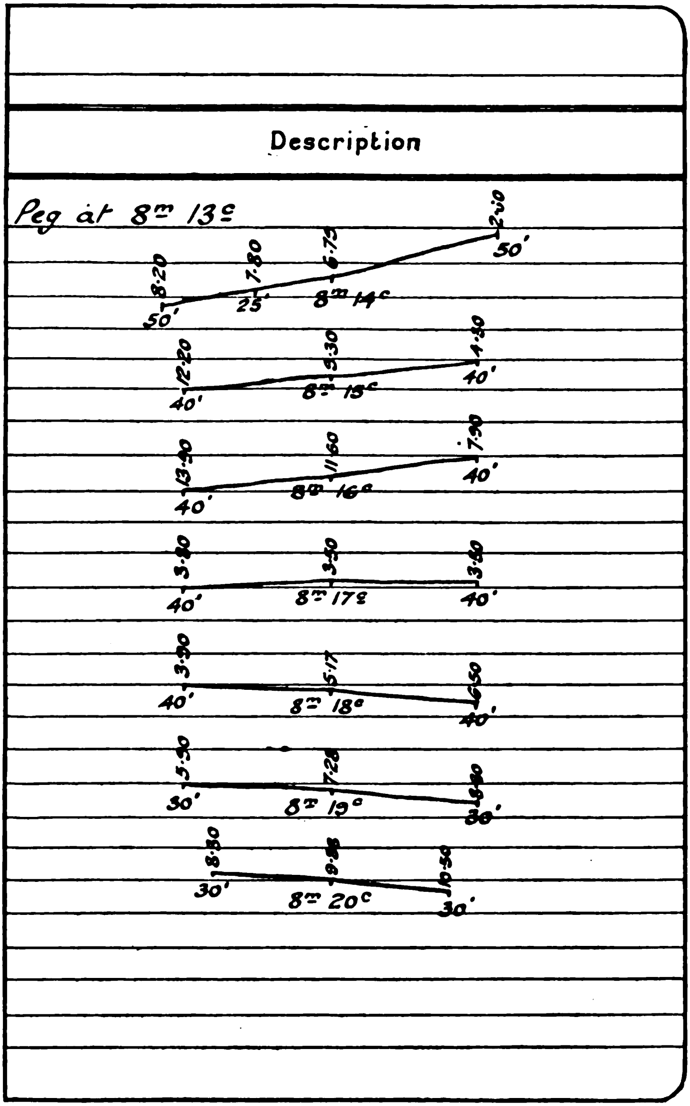
An adaptation of the mechanic's level furnishes a useful method for rapidly taking cross-sections on steep ground. A rod 5 ft. in



height is used with a cross-piece and pair of sights fixed on top accurately at right angles. A mechanic's level is attached to the

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cross-piece or laid on it, and indicates when the line of the sights is level. A mirror may be fixed so that the bubble may be seen while a sight is being taken. In taking a cross-section the rod is held on a levelled point of the longitudinal section with the cross-piece in the line of the cross-section. When the bubble is at the centre a sight is taken with the eye, and an assistant is directed to mark the point where the line of sight strikes the ground. This point is 5 ft. above the point on the longitudinal section, and the horizontal distance is measured with the tape. The appliance is brought up and held on this point and the taking of the cross-section is continued to the desired distance by successive vertical steps of 5 ft. and measured horizontal distances.

The above description applies to the taking of a cross-section working uphill. In proceeding downhill from the longitudinal section the appliance requires to be set up and a few trial sights taken till a point is found from which the line of sight strikes the higher point.

Cross-sections taken by this method in vertical steps of 5 ft. are very easily and rapidly plotted on cross-section paper.

If the cross-sections extend to some length and require to be accurate they must be taken with the level. In this method the levels of the points on the cross-sections are calculated in the usual manner and the cross-sections are plotted from a datum line as in the case of the longitudinal section, although they may also be plotted directly from the staff readings in the manner indicated below.

A concise graphical method of booking cross-sections which greatly facilitates plotting is illustrated on p. 269. The levels of the pegs and intermediate points on the longitudinal section are booked in the ordinary way, but for the cross-sections the staff reading on each point and the distance of the point from the centre-line peg are marked on a sketch. In plotting the cross-sections a separate base line is used for each, the points being plotted by laying off the staff readings downwards from the base line, as shown in Fig. 215. ABC is the temporary base line from which the ground surface line A′B′C′ is plotted by laying off the staff readings 2·60, 6·79, &c., vertically downwards to scale at the points at which the levels were taken. Point B′ represents the centre-line peg, and its level is written on the cross-section to serve as a datum for the

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plotting of the formation level, when the latter has been worked out from the longitudinal section. The arrangement of a series of cross-sections when plotted in this manner is shown in Fig. 216. A line representing the centre line of the railway is drawn either vertically on a sheet of paper of the usual size or along the middle of a narrow roll. The cross-sections are plotted at close intervals about the centre line, sufficient space being always allowed for the plotting of the embankment or cutting on each. The cross-sections are best plotted in consecutive order from the bottom to the top of the sheet or from left to right in the case of a continuous roll.

Long cross-sections will, as a rule, be levelled and booked in the manner described for longitudinal sections.

The labour of plotting cross-sections is diminished and the speed increased by the use of squared paper, known as "cross-section paper." The two sets of lines crossing each other at right angles are usually spaced either ten or twenty to the lineal inch, every fifth or tenth line being drawn heavier than the others as a guide in counting the spaces. The scale chosen for the cross-sections is made to suit the divisions of the paper. Cross-section paper is specially useful in plotting work directly from the staff readings.

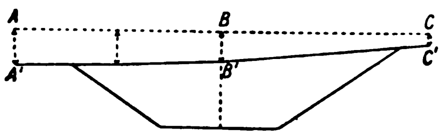

FIG. 215.—Plotting Cross-Section.

CONTOURS.

A contour is an imaginary line on the surface of the ground passing continuously through points of equal elevation. The water edge of a still lake is a contour line, and a contour line at any given elevation may be imagined as the shore line formed by still water flooding the earth up to that elevation.

Use of Contours.—Contours are usually plotted on survey maps and engineering plans at equal vertical height intervals. The interval may be 1, 2, 5, or 10 ft., or some such amount, in the case of plans to be used for purposes of designing and laying out of works and calculating quantities, while in the case of topographic maps giving a view of an extensive territory the interval may be 10, 20, 50, 100 or 200 ft., or more, depending on the scale and purpose of

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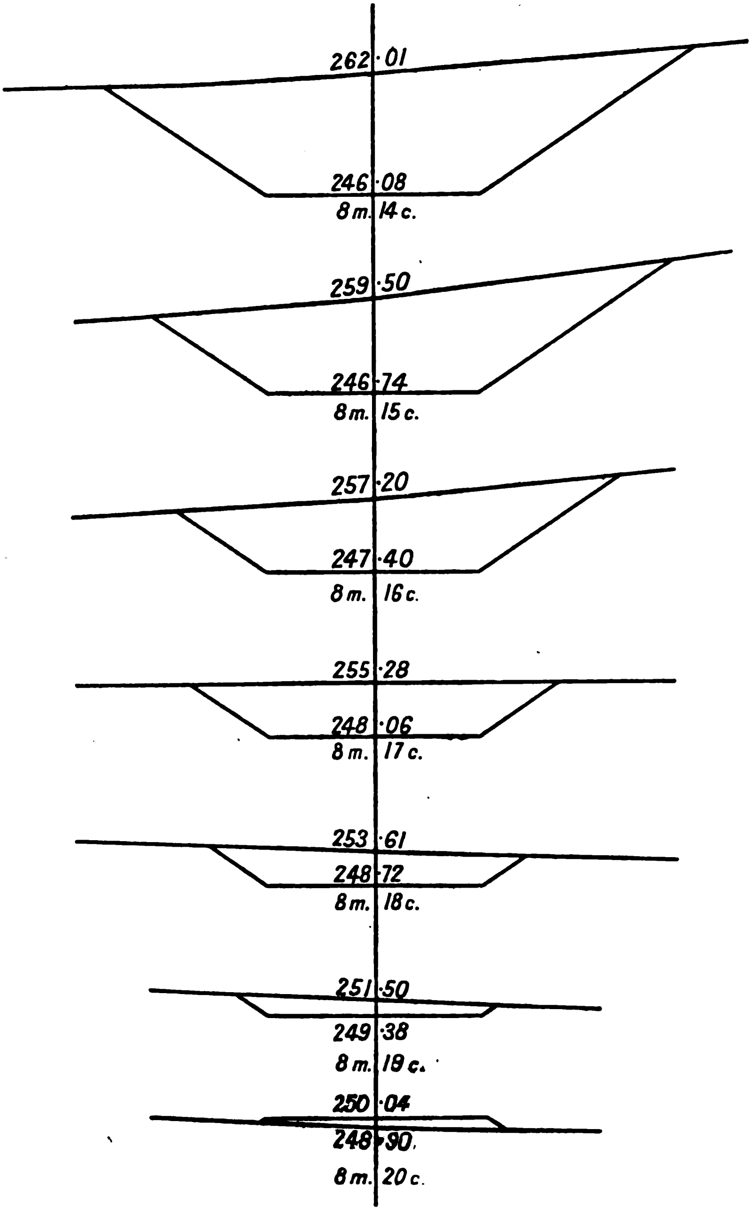

FIG. 216.—Series of Cross-Sections.

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the map. Contours plotted at equal height intervals show the form of the ground surface very graphically. The main points to be noted in reading the information conveyed by a contour plan are illustrated in Fig. 217.

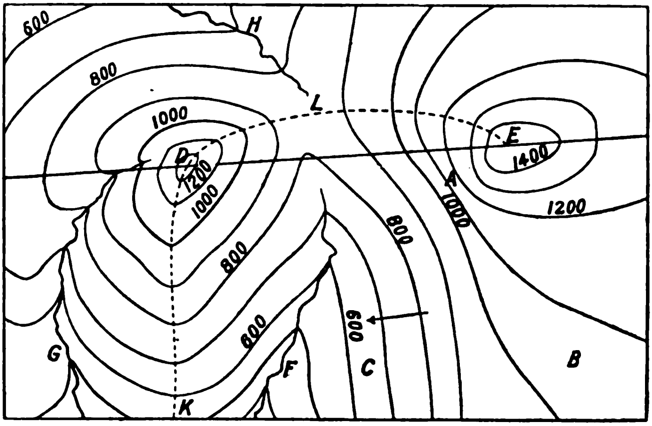

FIG. 217.—Typical Contour Plan.

The contours being at equal height intervals the steepness of the slope of the ground will be proportional to the closeness of the spacing of the contours on the plan. Thus in the figure the steepest slope

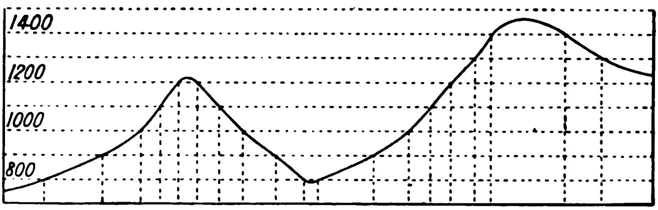

FIG. 218.—Section Plotted from Contour Plan.

is shown at A while at B the ground is comparatively flat. Uniform spacing of the contours represents a slope of uniform gradient as at C, and if in addition the contours are straight the slope will be a plane surface. The direction of maximum steepness at any point is at right angles to the contour as represented by the arrow at C.

Hills are represented by a series of closed contours of diminishing

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size and increasing altitude lying within each other as at D and E. A similar series with the altitude diminishing towards the smallest contour ring would represent a depression.

Valleys are shown at F, G and H, where the contour lines converge on either side towards the stream or valley bottom. Each contour thus forms a more or less acute angle pointing towards the head of the valley, and the elevation of the contours increases as you proceed in the direction in which the angles point.

A ridge, as shown by the dotted line KD, is practically the converse of a valley. The contours form a series of rounded angles pointing in a general direction along a straight or curving line, and taken in order in the direction in which they point they are of diminishing elevation.

A watershed is shown by the dotted line DLE. It passes from hill to hill at right angles to the contour lines and through the highest point L of the pass between the hills.

Fig. 218 shows a section along the line DE as plotted from the contours. To plot a section along any given line on a contour plan, a simple method of procedure is to lay the edge of a strip of paper along the line on the plan and mark off in pencil the points where it cuts the contours, and then transfer these points to the datum line of the section. Verticals are erected at each of these points, and the contour levels are then plotted giving points on the surface of the ground. Instead of plotting the level of each point separately, horizontal lines may be drawn at the levels of the contours, as shown in Fig. 218, the points on the surface being then given by the intersection of each vertical with its proper contour level.

The heights of the contours should be marked on the plans, as indicated in Fig. 217. It is not necessary to figure the level of every contour, but usually every second or fifth will be marked, the figures being arranged in continuous lines suitably distributed over the plan. The clearness of the plan will be enhanced by emphasising every fifth or tenth contour, either by drawing them in a heavy line, as in Fig. 217, or by using ink of a different colour. The contours emphasised will be those whose levels are the roundest multiples of 10.

The contours require to be carefully figured where depressions occur. Fig. 219 shows the contours of a conical hill with a volcanic

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crater on its summit, the true section across the summit being shown in Fig. 220. The hill might be taken to be of the form shown in Fig. 221, if the level of each contour near the summit were not carefully marked.

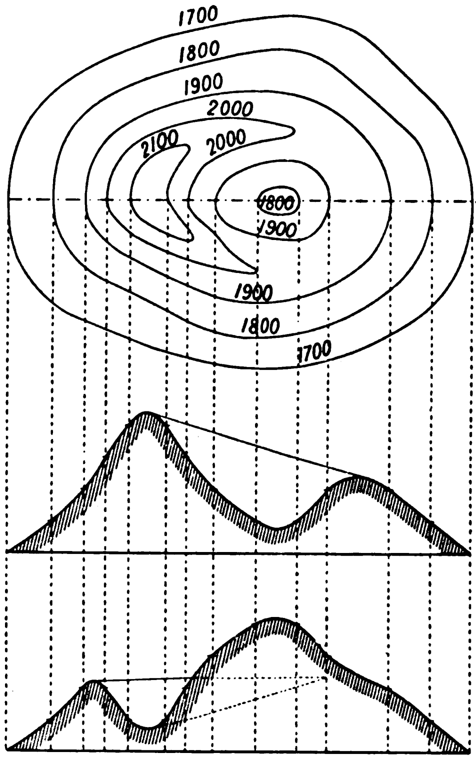

FIG. 219.

FIG. 220.

FIG. 221.

FIGS. 219, 220, 221.—Contour Plan and Sections of Volcanic Crater.

Some of the purposes for which contour *plans* are useful are the following :—

(*a*) Location of final routes of railways, roads, &c., and in the location and arrangement of works of engineering generally.

(*b*) Town planning and laying out of building areas.

(*c*) Calculation of capacity of reservoir.

(*d*) Calculation of quantities of earthwork.

Contour *maps* are useful as showing comprehensively the features

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of elevation of a large area, and for purposes of prospecting, locating approximate routes, determining extent of drainage areas, &c.

Locating Contours.—(*a*) By marking the contours out on the ground and then surveying them.

One method of procedure is to first set out a series of pegs at the contour heights along a line running, if possible, from the lowest ground to the highest, and then use these pegs as the starting points from which to set out points on the actual contours on the ground. On a large area several such series of pegs would be set out, and in any case it is advisable to have at least two lines, so that the accuracy of the contours may be checked by having them start on a peg on one line and finish on a peg at the same level on the other line.

In setting out the lines of pegs and the contours, the levels must be reduced as the work proceeds, as the height of instrument at each set-up requires to be known at the time. Assume that the levelling has commenced from a bench mark and that the instrument is now set up in position to commence the setting out of a line of pegs. The levels have been reduced, and the instrument height is found to be 58·34. Pegs are to be set out at vertical intervals of 5 ft., starting from elevation 50. To set the first peg at elevation 50 the staff must read 8·34 and the staff-holder is, therefore, directed up or down hill till the staff held on the ground gives this reading very nearly. The peg is then driven partly down and, finally, a little at a time, till the staff held on its top gives the correct reading. The peg at contour 55 may be got at the same setting of the level by driving it to give the reading 3·34 on the staff. The instrument would then be shifted uphill to a suitable position for setting out the peg at elevation 60 and one or two higher ones if they were within horizontal range. The instrument height is worked out for this position, and by subtracting 60 from it the staff reading for setting the peg at contour 60 is obtained, and so on for the other pegs.

To run a contour on the ground the level is set up in a suitable position to read on to the peg at the level of that contour, and the reading is taken on the staff held on the peg. This gives the height of the instrument above the contour and, therefore, any point on the ground which gives the same staff reading will lie on the contour. The staff-holder, therefore, walks along the contour as nearly as he can judge for, say, twenty or thirty paces and holds the staff on the

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ground. The observer at the level notes the reading, and directs the staff-holder uphill or downhill accordingly until the proper reading is obtained. The exact spot is then best marked for future location by sticking a piece of lath into the ground. A series of points is set out in this manner within the range of the instrument and a change point is then taken, the instrument is moved forward, the new instrument height is calculated from which the staff reading for the second series of points is obtained, and these are then set out and marked by laths. The running of the contour continues in this manner to any desired length, or until it checks on to another peg or completes a circuit and returns to the starting peg.

Where the contours are at close vertical intervals, and likewise at fairly close horizontal intervals, portions of two or more may be marked out at each setting of the level.

Where there is any likelihood of confusion in surveying the separate contours, owing to the closeness of the spacing, it is advisable to have several sets of laths painted in different colours or otherwise distinctively marked, and to run each single contour of one colour or marking throughout.

The same principles should govern the spacing of contour points as apply in the case of surveying a boundary. Where a portion of the contour is evidently a straight line on the ground it will suffice to mark the end points of that portion. Where the contour is curving somewhat regularly, points will be taken at about equal intervals measured by pacing. The points must be taken close together where the contour is bending sharply, and all abrupt changes of direction should be carefully located. For a large scale plan the points to locate the contours should throughout be taken at closer intervals than for a small scale plan.

The surveying of the contours after they have been run out might be effected by any of the methods of surveying which have already been described, and which may be suitable to the circumstances. The surveying of contours on the ground would, however, usually only be undertaken where they occurred at wide intervals, and method (*b*) next described would generally be preferable for the location of contours.

(*b*) By taking spot levels or cross-sections, locating their positions, and subsequently, in the office, deducing and plotting the positions of the contours.

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Fig. 222 shows an area with the cross-sections and levels which have been taken for the purpose of plotting the contours. The levels of the points may be marked simply in pencil. The contours will seldom happen to pass through any of the points whose levels have been taken. The position of a contour between two levelled points is fixed by assuming that the ground has a uniform gradient between the points. The contour must then divide the horizontal distance between the points in the same proportion as it divides the vertical distance. Where the distances between points on the plan are not great the positions of the contours are usually interpolated by estimation.

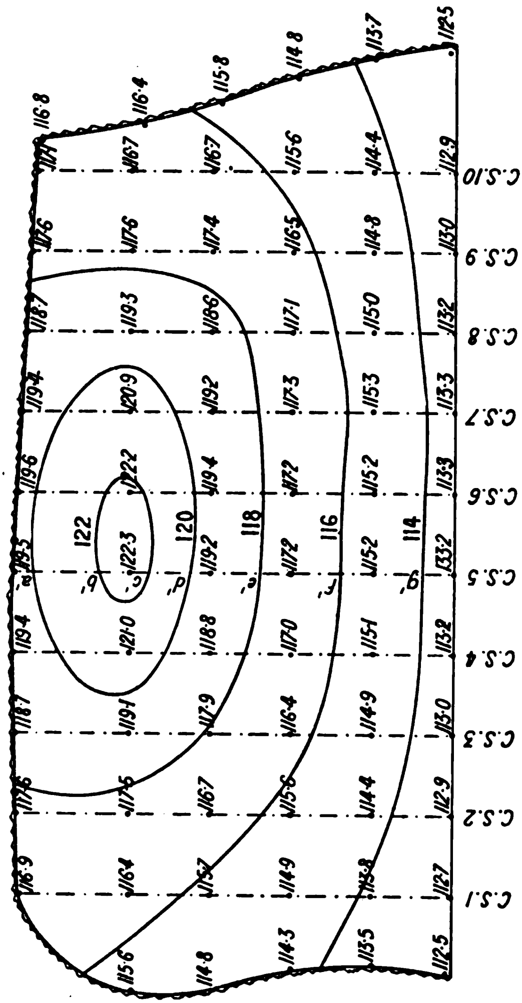

FIG. 222.—Cross-sections for Contour Purposes.

An accurate method of finding the positions of the contours on the cross-sections is by plotting the latter and drawing horizontal lines at the elevations of the contours. The intersections of these horizontal lines with the ground surface line give the positions of

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the contours, and these may be transferred to the plan. This method is illustrated in Fig. 223, which shows cross-section No. 5 on the preceding figure plotted from the levels. Points *a*, *b*, *c*, *d*, &c., where the contour elevations meet the ground line, enable the corresponding positions of the contours to be plotted on the plan.

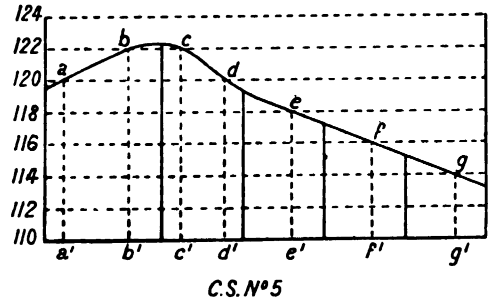

FIG. 223.—Finding Positions of Contours.

Contours may be rapidly and accurately interpolated between the levelled points by the employment of graphical methods. In the method illustrated in Fig. 224, a system of equally spaced parallel lines is drawn on tracing cloth, every tenth line being emphasised. The thick lines on this diagram are numbered to represent contours with an interval of 1 ft. It is required to interpolate the contours between two points, A and B, whose elevations are respectively 111·4 and 114·3. The thin line numbered 1·4 on the diagram would be placed over point A, the tracing would be held down with the finger or with a pricker at point A and rotated till the line numbered 4·3 passes through the point B. It will be evident that the thick lines numbered 2, 3, and 4, then cut the line joining A and B in the positions of the contours 112, 113, and 114, and these points may be pricked through on to the plan.

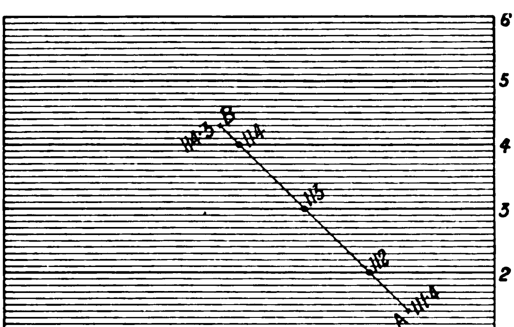

FIG. 224.—Finding Positions of Contours.

The lines on a diagram, such as the above, need only be temporarily numbered in pencil to suit the particular work in hand.

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For large scale plans the heavy lines may represent single feet, while for smaller scales they may represent intervals of 2, 5, 10 or more feet, as may be found convenient in plotting.

Another graphical device employing lines drawn on tracing cloth is illustrated in Fig. 225. There is a radiating system of lines with every tenth line emphasised, and crossing this system there is a series of parallel guide lines. In using the diagram the guide lines must be kept parallel to the line joining the two points on the plan. To place it in position for giving the contours between the points

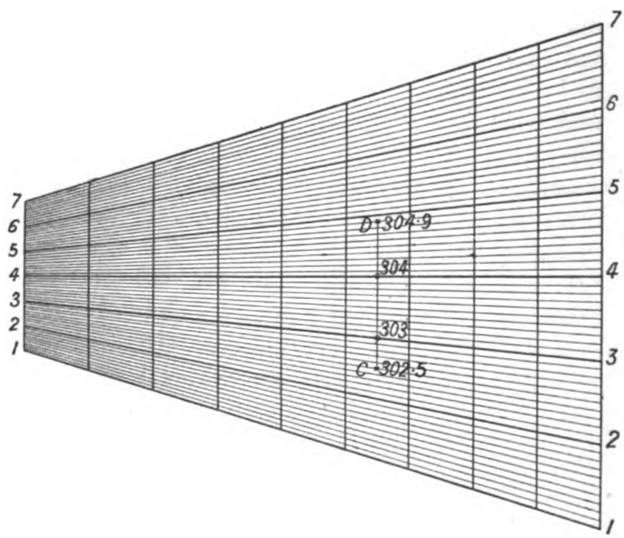

FIG. 225.—Finding Positions of Contours.

C and D, the line numbered 2·5 would be placed over point C and the diagram disposed so that the guide lines are parallel to the line CD. The diagram would then be moved to the right or left without rotation, keeping line No. 2·5 over point C, till the line No. 4·9 passes through point D. Line CD will then be intersected by the heavy lines of the diagram in the positions of the contours.

Use of Contour Plans.—(*a*) Location of route. A simple case of the use of a contour plan in the designing and location of engineering works is illustrated in Fig. 226. The problem is to lay down on the plan from point A a suitable route for an outfall sewer

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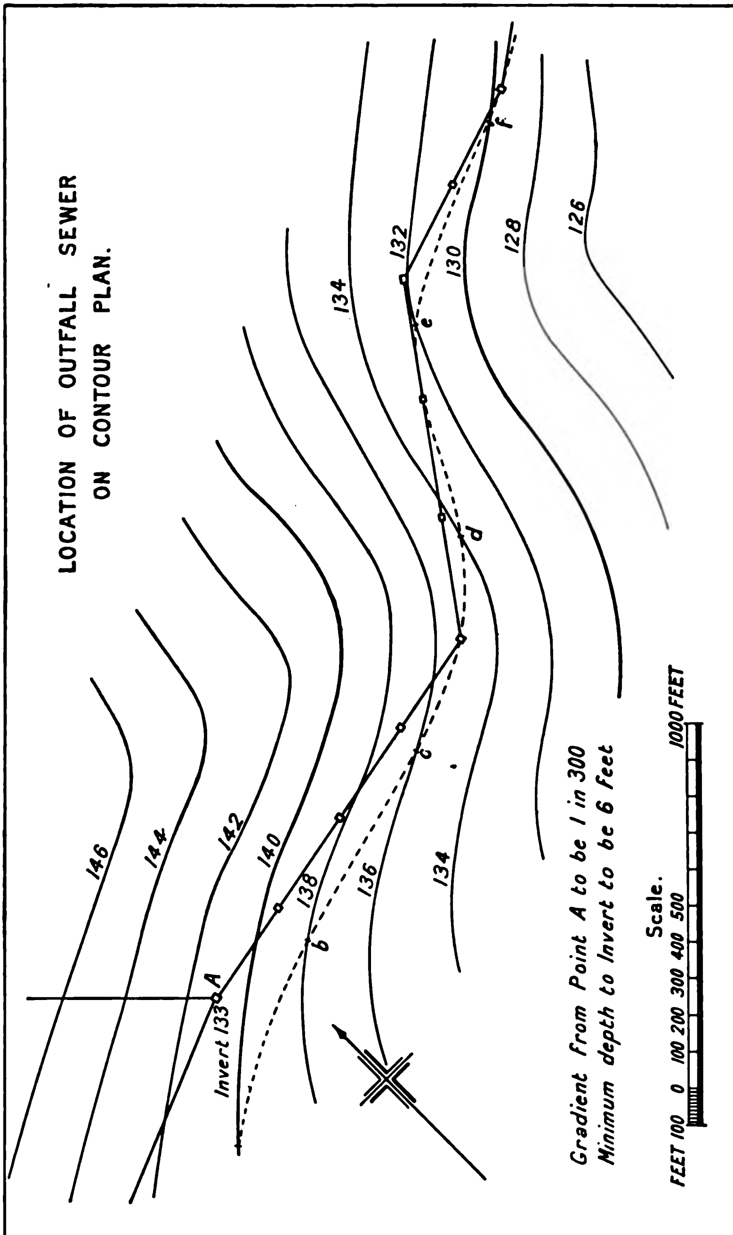

FIG. 226.—Use of Contour Plan in Designing.

with a gradient of 1 in 300 and a minimum depth of 6 ft. to the invert of the pipe. The sewer is to be in straight lengths between manways which are to be formed at intervals of about 100 yards.

The level of the invert at A is 133 ft. above datum and the level of the ground surface is about 141, so that the depth at this point is about 8 ft., that is, 2 ft. more than the desired depth of the outfall sewer. The first thing to be done is, therefore, to arrange the position of the first manway, at a distance of about 100 yards from the junction chamber, so that it will have a depth of about 6 ft. In a distance of 300 ft. the sewer will have a fall of 1 ft., so that the invert level at the first manway will be 132, and the ground surface level should, therefore, be about $132 + 6 = 138$. If we, therefore, take a distance of 300 ft. on the dividers, and, placing one end at A find where the other point cuts the 138 contour line (point *b*) this will fix the probable position of the manway.

The contours are at vertical intervals of 2 ft. A fall of 2 ft. in the sewer will take place in a distance of 600 ft., so that if an ideal route were set out with a depth everywhere of exactly 6 ft. the line of the sewer would cut the contour lines at uniform intervals of 600 ft. An approximation to the ideal route will be obtained if we take a distance of 600 ft. on the dividers to the scale of the plan and, starting from point *b*, step successively from contour to contour, thus fixing points *c*, *d*, *e*, *f*. The ideal route may now be sketched in, and is represented by the dotted line on the plan. The actual route to satisfy the requirements of the case will be a succession of straight lines approximating as closely as may be to the ideal route and arranged with manways at all changes of direction and at other intermediate points where necessary. The full line on the plan shows the line of the sewer as designed, the small squares indicating the positions of the manways.

(*b*) Laying out of building areas, &c. The use of a contour plan is almost a necessity if the roads and streets of a proposed building area are to be laid out to the best advantage. The question of suitable drainage arrangements for the area is intimately connected with the scheme of lay-out of the roads, so that the features of elevation of the entire area must be considered. A contour plan gives the information required as to these features, and when the contours are at equal vertical intervals and fairly close together we have the

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information in a very expressive form. The level portions of the ground, steep slopes, summits of hills, bottoms of valleys, and direction of natural drainage are all seen at a glance. The good or bad features of any proposed arrangement of roads will also be evident if a tracing of the scheme, prepared to the scale of the contour plan, is applied over the latter. When an arrangement of roads has been adopted, longitudinal sections of the roads may be at once plotted from the contours with sufficient accuracy to enable the gradients to be finally fixed, the amounts of cutting and embankment to be approximately determined, and the sewers and drainage arrangements to be designed.

(*c*) Calculation of quantities. To illustrate the use of contours in the calculation of quantities we shall take the case of a reservoir formed behind an earthen embankment which closes in a valley. Contours of the area which will be flooded have been plotted on a plan at vertical intervals of 5 ft. Every contour which occurs below the level of the top of the embankment in this area will form a circuit passing along the inner face of the embankment and up both sides of the valley to meet on the line of the stream. We may imagine a series of level surfaces at vertical intervals of 5 ft. passing through the contours and dividing up the capacity of the reservoir into horizontal slices 5 ft. thick. The areas of the upper and lower surfaces of any slice will be the areas contained within the bounding contours, and the volume of the slice will be approximately obtained by multiplying the mean of the two areas by the thickness of the slice. The total capacity of the reservoir will be got by adding together the capacities of the separate slices obtained in this way. The capacity of the reservoir when the water is standing at any given level is required as well as the total capacity, and in the example on p. 284, the figures are arranged so as to give this information. The bottom of the reservoir is below the contour level 145, but the calculation of the capacity commences at level 150, being the lowest point from which water can be drawn off for use.

The contour areas may be calculated by the method of sub-division into geometrical figures, but in a case like this where we have a series of concentric areas with irregular outlines the employment of the planimeter will furnish the most expeditious method, provided the plan is not large.

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Calculation of Capacity of Reservoir.

Contour.	Area.	Mean Area.	Capacity of Layer.	Capacity of Reservoir from Elevation 150 up to Elevation.		
	Sq. Feet.	Sq. Feet.	Cub. Ft.	Cub. Ft.	Gallons.	
145	11,240					
150	75,380					
		114,790	573,950			
155	154,200			573,950	3,587,000	155
		196,950	984,750			
160	239,700			1,558,700	9,742,000	160
		281,050	1,405,250			
165	322,400			2,963,950	18,524,000	165
		357,600	1,788,000			
170	392,800			4,751,950	29,699,000	170
		425,100	2,125,500			
175	457,400			6,877,450	42,984,000	175
		498,250	2,491,250			
180	539,100			9,368,700	58,555,000	180
		587,400	2,937,000			
185	635,700			12,305,700	76,910,000	185
		685,950	3,429,750			
190	736,200			15,735,450	98,346,000	190
		788,700	3,943,500			
195	841,200			19,678,950	122,993,000	195
			19,678,950			

Levelling Problems and Devices.

Levelling on Steep Slope.—In levelling up a steep slope, as illustrated in Fig. 227, the backsights will usually read nearly to the top of the staff and the foresights nearly to the bottom, and if the level is set up approximately in line between the change points the backsights will be on the average about twice as long as the foresights, and, therefore, error due to curvature will, as already explained, accumulate. The difference in length between the backsights and foresights and the liability to error may be greatly reduced by setting up the level, not in line between the change points but at a considerable distance to the side, at points such as G and F in plan (Fig. 228), instead of points E′ and D′.

Levelling over Summits and Hollows.—Time and labour may be

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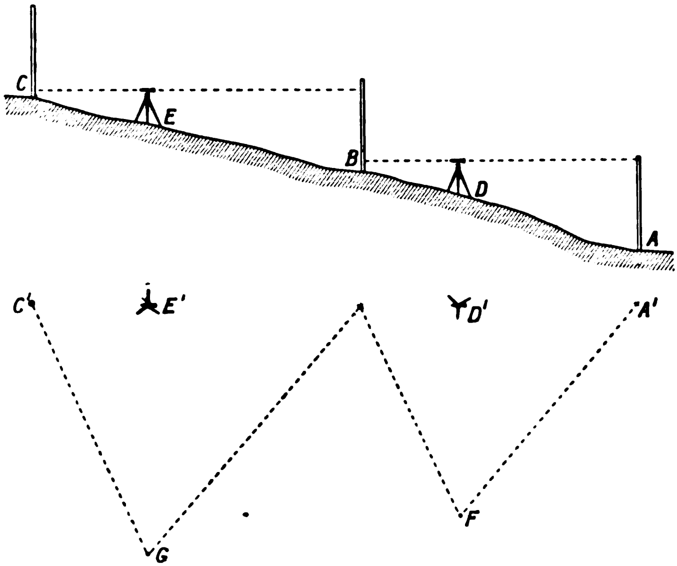
economised by taking the precaution of only setting up sufficiently high to see over a summit, as illustrated in Fig. 229, the natural inclination being to set up right on the summit, a proceeding which



FIGS. 227 AND 228.—Levelling up Steep Slope.

might necessitate an extra change. The corresponding precaution in crossing a hollow of setting up only sufficiently low to enable the levels of all the required points to be obtained, as shown in Fig. 230, should also be observed.

Taking Level of an Overhead Point.—The level of an overhead point may be obtained by holding the inverted staff on it, as shown in Fig. 231, and taking the reading, which will give the height of the point above the line of sight. The reading must, therefore, be added to the height of the instrument to give the reduced level of the point, and should be entered in the level book marked with a *plus* sign. So that there may be no opportunity for mistake it is well also to make a note on the description page that the staff has been held inverted. Thus, for the case shown, the entry in the

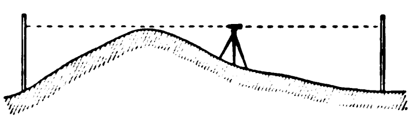


FIG. 229.—Levelling over Summit.

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column of intermediate sights would be + 10·27, and the relative description on the right-hand page "on underside of north girder at centre, staff inverted."

Obstructions.—Most obstacles of the nature of obstructions to the line of sight can be overcome by levelling round about, but a little ingenuity expended in devising a simple expedient will often economise time and labour.

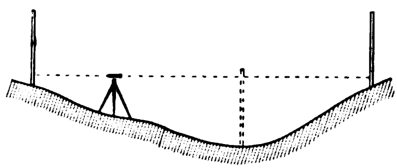

FIG. 230.—Levelling across Hollow.

To level past an obstruction such as a close-boarded fence, which can neither be seen over nor seen through, we may either drive a nail through a board near the bottom so as to project on both sides, or we may pass an arrow or a knife blade through one of the joints. In each case we obtain two points at the same level, one on each side of the fence, and they may be considered as one change point. The staff will be held for the foresight on the nail, arrow, or blade on one side of the fence and the level and staff will then be taken round, the level will be set up, and the backsight will be taken to the staff held on the portion of

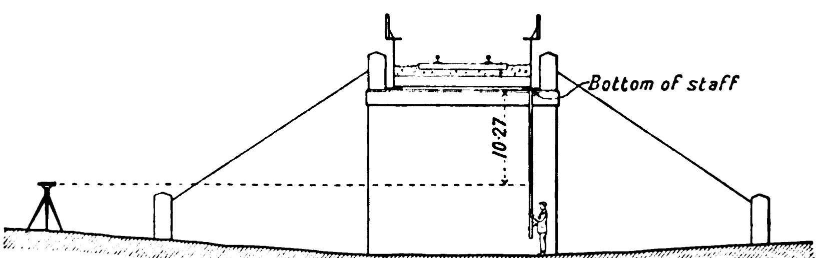

FIG. 231.—Level of Overhead Point.

the nail, &c., which projects inside. The readings will be booked as for a change point in the ordinary way.

In the case of a high brick wall we may fix on a horizontal joint near the bottom as a bench mark. Its position can be identified by counting the number of courses down from the top, and the same joint on the other side of the wall can be found by counting down the same number of courses. These two points at the same level

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will then be considered as one change point, the foresight being taken to the staff held on the joint on one side of the wall and the backsight to the staff held on the same joint on the other side. In some cases it may be better to fix points at the same level by

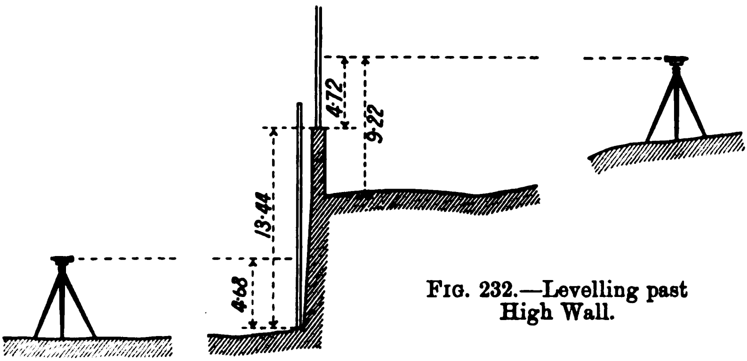

FIG. 232.—Levelling past High Wall.

measuring down equal distances from the top of the wall or from the under edge of the cope on each side.

The foregoing methods are not applicable to the case shown in Fig. 232. A method of booking the readings and measurements and obtaining the levels in the case of such an obstruction occurring in a longitudinal section is given below.

Back.	Inter.	Fore.	Ht. of Inst.	Reduced Level.	Dist.	Description.
—	4·95	—	243·66	238·71	1,300	
—	—	4·68	—	238·98	1,366	At base of wall, on stone.
—	—	—	—	252·42	—	13·44 above stone to top of wall T.B.M.
4·72	—	—	257·14	252·42	—	On T.B.M. on top of wall.
—	9·22	—	—	247·92	1,368	At base of parapet.
—	8·89	—	—	248·25	1,400	

Crossing Pond or Lake.—In crossing a pond or lake advantage may be taken of the fact that the surface of still water is a level

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surface. A reading may, therefore, be taken to the staff held at the water level on one shore as a foresight and on transferring the level to the opposite side, a backsight may be taken to the water level at any point of the shore. For the purpose of reducing the levels the foresight and backsight may be considered as taken on the same change point.

In the case of a flowing river levels may be continued from one side to the other in the above manner with little error, provided care is taken to choose a comparatively still stretch and to see that the water levels are taken at points directly opposite each other.

Reciprocal Levelling.—This is a method of levelling by which the true difference of elevation of two points a considerable distance

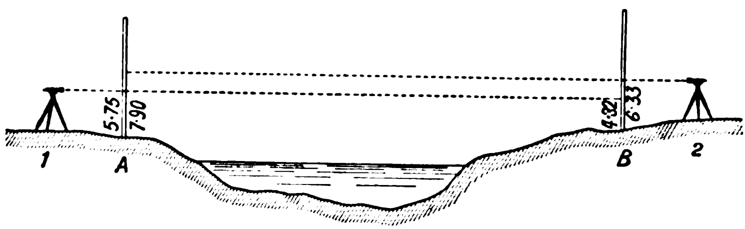

FIG. 233.—Reciprocal Levelling.

apart may be obtained by two sets of observations. The process employed automatically corrects for error in adjustment of the level and for the effect of curvature and refraction. It is applicable to cases such as the crossing of a wide river, where there is no bridge to enable the levelling to be continued across in the ordinary manner.

Referring to Fig. 233, readings are taken with the instrument set up at point 1 to the staffs held on points A and B. The level is then transferred to point 2, the distance 2B being arranged practically equal to the distance 1A, and readings are again taken to the staffs held at A and B. The readings taken from position 1 will give a certain difference of elevation, say, D_1, between the two points, and the readings from position 2 will give another difference of elevation, say, D_2. The true difference of elevation is the mean of the two determinations, that is $= \dfrac{D_1 + D_2}{2}$.

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Suppose that the readings on the staffs were as given below—

	From Position 1.	From Position 2.
Reading on A	5·75	7·90
,, B	4·32	6·33
	Diff. $D_1 = 1·43$	Diff. $D = 1·57$

Actual difference of elevation $= \dfrac{1·43 + 1·57}{2} = 1·50$, that is, point B is 1·50 ft. higher than point A.

Contour Grading.—The operation of running a line on the surface of the ground on a constant gradient along the face of sloping ground is known as "contour grading." It is frequently required in connection with the location of roads, railways, and water conduits. The operation may be performed with level and staff, in which case points would be marked on the ground at equal intervals in somewhat similar manner to that described for running a contour, the difference being that the successive staff readings from one setting of the instrument would not be equal, but would increase or decrease according to the gradient, thus on a falling contour gradient of 1 in 100, points might be marked at 100-ft. intervals, the staff reading being increased by 1 ft. at each interval. As in running contours, the levels require to be reduced as the work proceeds.

Contour grading will usually be more easily and expeditiously accomplished with the theodolite than with the level. The theodolite is set up on the ground on the line of the gradient at its commencement, and levelled and adjusted as for reading a vertical angle, the bubble of the telescope level being brought to the centre with the vertical circle index reading zero (see p. 35). The telescope is then tilted up or down to read a vertical angle equivalent to the gradient, thereby setting the line of sight to the required inclination. For gradients not steeper than 1 in 10 the vertical angle for a gradient $\frac{1}{n}$ will be given very closely by the formula —

$$\text{Vertical angle} = \frac{3438}{n} \text{ minutes.}$$

Thus for a gradient of 1 in 75 the angle would be $= \dfrac{3438}{75} = 45·8$.

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minutes. The height of the telescope above the ground is taken on a staff or ranging rod, the telescope is sighted along the face of the slope in the direction in which the gradient is to run, and points are marked out as for an ordinary contour to give this constant reading on the staff. The reading is usually taken to the hand of the assistant held at the proper point on the staff or rod.

The setting out of a contour gradient line from one position of the theodolite is continued only for such distance as the general direction of the line remains constant. Where the line takes a bend, and at other places where the conditions of view require it, a point will be set out on the gradient line, similar to a change point in levelling, the theodolite will be brought forward and set up over this point, and the setting out of the gradient line ahead will continue as before.

The Abney level, described on p. 23, with the index set to the required angle, may be used in similar manner to the theodolite for rapidly running a rough trial gradient line. Readings are taken to a height of staff equal to the height of the eye above the ground.

A contour gradient set out on the ground and marked with laths will usually appear as an irregular winding line. In the case of roads and water conduits, where curves of large radius are not required, the final centre line may often be set out directly from the contour gradient. Mean straight lines would be set out to replace those portions of the gradient line which keep a fairly constant general direction, and the various straights would be connected by curves of suitable radius.

In the case of a railway, where the curves must always be of large radius (seldom less than 500 ft. for a very subsidiary line), the fixing of the best location for the centre line is not a simple matter. On hilly ground the usual procedure is to run a traverse survey, following the gradient line as closely as possible, and read the intersection angles. A section is taken with the level along the lines, and sufficient cross-sections are taken with the inclinometer or Abney level to determine the general form of the ground for some distance on either side of the traverse lines. The fixing of the best location for the centre line is subsequently a matter of office work. The traverse is plotted on paper, and from the cross-sections the actual contours are laid down. These enable a longitudinal section along any line to be plotted, and it may be necessary to plot sections

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on one or two trial lines before the most satisfactory location is attained. When a final centre line has been fixed on the paper, it is set out on the ground, as regards the straights, by measurements from the traverse lines, the curves are run in by theodolite, as described in Chapter XIX., and accurate levelled longitudinal and cross sections are taken. These latter are used for the preparation of the working plans.

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CHAPTER XIX

SETTING OUT CURVES, ETC.

In this chapter some methods of setting out circular curves on the ground will be considered. Methods involving the use of the chain and tape alone, or of the chain, tape and optical square, may be adopted in certain simple cases where great accuracy is not necessary or where a theodolite is not at hand. For curves which extend to considerable lengths, and which require to be accurately set out, the use of the theodolite is almost imperative. The setting-out of building work is treated briefly.

Methods requiring Use of Chain, Tape and Optical Square.—The following methods of setting out curves do not require the use of the theodolite:—

(*a*) By offsets scaled from a plan.
(*b*) By a radius swung from the centre of the curve.
(*c*) By perpendicular offsets from a tangent line.
(*d*) By offsets from chords produced.

Method by Offsets scaled from a Plan.—This method is applicable to a circular curve or to any form of curve which has been accurately drawn on a plan to a sufficiently large scale. A base line or series of base lines is drawn on the plan to comply with the following conditions: The offsets should be as short as possible and the base lines should be chosen so as to be easily set out on the ground. The positions of the base lines are fixed by measurements scaled off the plan to definite points of existing objects which can be readily located on the ground, and by setting out these measurements on the ground a reproduction of the base lines as drawn on the plan will be obtained. Points are marked off by scale at equal distances along the base lines on the plan and the lengths of perpendicular offsets to the curve at these points are scaled off. These distances

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and offsets being set out along and from the base lines on the ground, a series of points will be obtained lying on the required curve. This method would be applicable to the setting out of a scheme of winding roads for a park or pleasure ground where great accuracy as to position was not essential, or to any similar case.

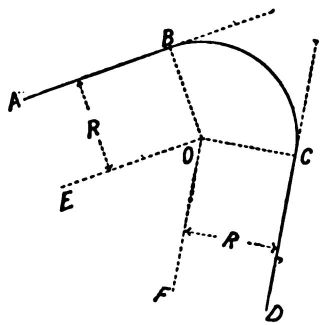

FIG. 234.—Setting out Curve by Radius.

Method by a Radius swung from the Centre of the Curve.—The general problem will be to connect up two straight lines tangentially by a circular curve of given radius. In Fig. 234, AB and DC represent the two straight lines which are to be joined up by a curve of radius R. The solution of the problem consists in finding point O, the centre of the curve, on the ground. Set out the line EO parallel to line AB and at a distance R from it, and similarly set out FO parallel to and at distance R from DC. Point O where the lines EO and FO intersect is the centre of the curve. The lines OB and OC set out at right angles to the lines AB and DC fix the tangent points B and C of the curve. The curve may be marked out on the ground between B and C by swinging a length of chain equal to radius R about the centre O. The method is only useful for curves of very small radius.

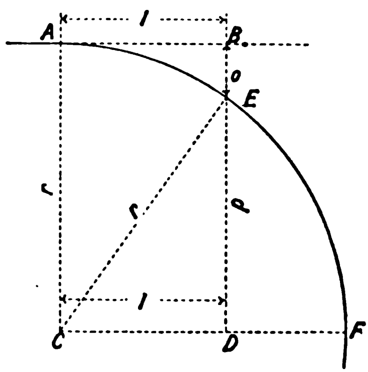

FIG. 235.—Offsets to Curve from Tangent.

Method by Perpendicular Offsets from a Tangent Line.—In Fig. 235, AB is a tangent to the circle AEF at the point A. C is the centre of the curve and CF is a radius parallel to AB. BE is the offset of length o from the tangent line to the curve at a point distant l from A, and DE is the corresponding offset of length p from the radius.

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In the right-angled triangle CED we have

$$p^2 = \quad r^2 - l^2$$
$$\text{or } p = \sqrt{r^2 - l^2}$$
$$\text{But } o + p = r \text{ or } o = r - p.$$
$$\text{Therefore } o = r - \sqrt{r^2 - l^2}$$

For ease of calculation the distances l should be taken in round numbers of units. A table of square roots will greatly expedite the work of calculation.

Example. A portion of a curve of thirty chains radius is to be set out by perpendicular offsets at intervals of one chain from the tangent point. The figures for the calculation of the first eight offsets are shown in the following table :—

$r^2 = 900.$			$r = 30{\cdot}0000$ chs.		
	l^2	$r^2 - l^2$	$\sqrt{r^2 - l^2}$	$r - \sqrt{r^2 - l^2}$	Length of Offset.
Chains			Chains.	Chains.	Links.
1	1	899	29·9833	0·0167	1·67
2	4	896	29·9333	0·0667	6·67
3	9	891	29·8496	0·1504	15·04
4	16	884	29·7321	0·2679	26·79
5	25	875	29·5804	0·4196	41·96
6	36	864	29·3939	0·6061	60·61
7	49	851	29·1719	0·8281	82·81
8	64	836	28·9137	1·0863	108·63

If the whole length of the curve to be set out forms only a very flat segment of a circle so that the last offset BC (Fig. 236) is not more than one-seventh of the length of l then the offsets may be set out as the ordinates of a parabola, and the resulting curve will hardly differ appreciably from a circle. The parabolic offsets are sometimes adopted in such a case because they are much more easily calculated than the circular offsets. For a parabolic curve the length of the offset is proportional to the square of the distance along the tangent from the tangent point or,

$$\text{Length of offset} = cl^2,\ c \text{ being a constant.}$$

Example : A straight line AB (Fig. 236) has been set out, and it is required to continue from the tangent point A along a curve

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which shall pass through point C. Lengths AB and BC measure 463 ft. and 62·5 ft. respectively. The curve is to be set out by offsets at intervals of 50 ft. measured along the tangent from A.

$$\text{Length of first offset } o_1 = 62{\cdot}5 \times \frac{50^2}{463^2} = 0{\cdot}7289 \text{ ft.}$$

The length of the first offset having been found as above the rest are worked out as in the following table :—

o_1 at 50 ft. = 0·7289 ft.	
o_2 ,, 100 ft. = 0·7289 × 4	= 2·92 ft.
o_3 ,, 150 ft. = 0·7289 × 9	= 6·56 ,,
o_4 ,, 200 ft. = 0·7289 × 16 = o_2 × 4	= 11·66 ,,
o_5 ,, 250 ft. = 0·7289 × 25 = $o_1 \times \frac{100}{4}$	= 18·22 ,,
o_6 ,, 300 ft. = 0·7289 × 36 = o_3 × 4	= 26·24 ,,
o_7 ,, 350 ft. = 0·7289 × 49	= 35·72 ,,
o_8 ,, 400 ft. = 0·7289 × 64 = o_4 × 4	= 46·65 ,,
o_9 ,, 450 ft. = 0·7289 × 81 = o_3 × 9	= 59·04 ,,
o_{10} ,, 463 ft. = $0{\cdot}7289 \times \frac{463^2}{50^2}$	= 62·5 ,,

It will be seen that most of the values can be obtained by short multiplication.

The setting out of offsets becomes laborious and unsatisfactory when they exceed the length of the chain or tape. When the offsets for a circular curve become too long a new tangent line can be set out, as shown in Fig. 236. The line EC will be a tangent to the circle at C if the distance EC is equal to the distance AE. The point E will lie a little beyond the centre of the length AB measuring from A and if D is the centre point the distance DE is equal to $\frac{o^2}{2l}$. The point E being set out to this distance the line EC produced will give a new tangent line CF from which the same series of offsets may be set out as from the

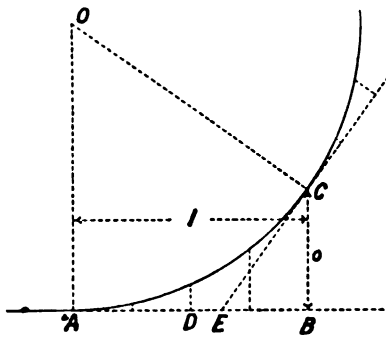

FIG. 236.—New Tangent.

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tangent AB. Any required length of curve can be set out from successive tangent lines in this manner.

In the above offset method where equal distances are measured along the tangent line the pegs set out along the curve will be at varying distances apart. If it is required to set out points at equal intervals apart along the curve the offsets will require to occur at decreasing intervals from the tangent point. Referring to Fig. 237, A is the tangent point and O is the centre of a curve of radius R. Points 1, 2, &c., are to be set out at equal distances l along the curve by offsets from the tangent line. The calculation requires the use of trigonometry, and the first thing to do is to find the size of the angle a which an arc of length l subtends at the centre of the circle, that is, the angle AO1 in the figure.

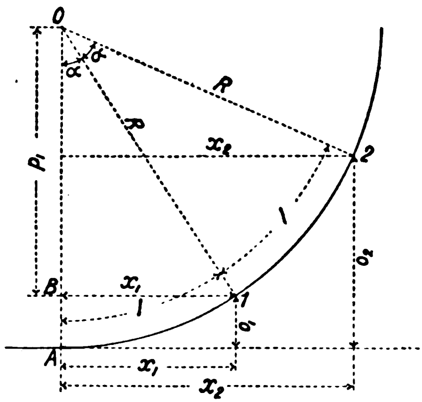

FIG. 237.—Offsets at Equal Curve Intervals.

$$\text{Angle } a = \frac{l}{R} \text{ radians} = \frac{l}{R} \times \frac{360}{2\pi} \text{ degrees.}$$

$$= \frac{l \times 360 \times 60}{R \times 2 \times 3.1416} \text{ minutes} = \frac{l}{R} \times 3438 \text{ minutes.}$$

In the triangle BO1 we have

$$\frac{x_1}{R} = \sin a, \text{ or } x_1 = R \sin a,$$

$$\text{similarly } x_2 = R \sin 2a.$$

Also we have $p_1 = R \cos a$, but $o_1 + p_1 = R$.

Therefore $o_1 = R - p_1 = R - R \cos a = R\,(1 - \cos a)$.

similarly $o_2 = R\,(1 - \cos 2a)$.

Example. Calculate the co-ordinates to set out six points at intervals of 100 ft. from the tangent point along a curve of 1,800 ft. radius.

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Central angle for arc of 100 ft. $= \frac{100}{1,800} \times 3,438$ minutes $= 191$ minutes or $3^\circ\ 11'$.

CALCULATION OF CO-ORDINATES FOR POINTS EQUIDISTANT ALONG THE CURVE.

Log R = 3·25527.			
Angle.	Log Sin.	Log x.	x.
$a_1 = 3^\circ\ 11'$	8·74454	1·99981	$100{\cdot}0 = x_1$
$a_2 = 6^\circ\ 22'$	9·04489	2·30016	$199{\cdot}6 = x_2$
$a_3 = 9^\circ\ 33'$	9·21987	2·47514	$298{\cdot}6 = x_3$
$a_4 = 12^\circ\ 44'$	9·34324	2·59851	$396{\cdot}7 = x_4$
$a_5 = 15^\circ\ 55'$	9·43813	2·69340	$493{\cdot}6 = x_5$
$a_6 = 19^\circ\ 06'$	9·51484	2·77011	$589{\cdot}0 = x_6$

	Log. Cos.	Log p.	p.	$o = 1800 - p$.
$a_1 = 3^\circ\ 11'$	9·99933	3·25460	1797·2	$2{\cdot}8 = o_1$
$a_2 = 6^\circ\ 22'$	9·99731	3·25258	1788·9	$11{\cdot}1 = o_2$
$a_3 = 9^\circ\ 33'$	9·99394	3·24921	1775·0	$25{\cdot}0 = o_3$
$a_4 = 12^\circ\ 44'$	9·98919	3·24446	1755·7	$44{\cdot}3 = o_4$
$a_5 = 15^\circ\ 55'$	9·98302	3·23829	1731·0	$69{\cdot}0 = o_5$
$a_6 = 19^\circ\ 06'$	9·97541	3·23068	1700·9	$99{\cdot}1 = o_6$

The calculations are made by means of logarithms, and the figures may be set down as in the above tables. The figures in the column headed "Log sin" are obtained from tables of logarithmic sines of angles. By adding the value of log R to each of the log sines we get the figures in the column headed "Log x" which are the logarithms of x_1, x_2, x_3, &c. The values of x_1, x_2, x_3, &c., are obtained by looking up from the tables the numbers which correspond to these logarithms.

The calculations for finding the values of the offsets o_1, o_2, o_3, &c., are as given in the lower portion of the table.

Kröhnke's curve tables give values of co-ordinates for setting out equidistant points, worked out for a large range of curves.

Method by Offsets from Chords produced.—In Fig. 238, O is the centre of a circle of radius R, and AB, BC, CD are successive equal chords

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of length l. Let AB be produced to E so that AB = BE. Then the triangle CBE is an isosceles triangle and similar to the triangle OBC so that we have $\frac{\text{CE}}{\text{CB}} = \frac{\text{CB}}{\text{OB}}$

$$\text{or CE} = \frac{\text{CB}^2}{\text{OB}} = \frac{l^2}{\text{R}} = \text{length of so-called offset } o.$$

This geometrical property furnishes us with a method of accurately setting out a circle. The first chord AB is produced its own length to E. On BE as a base a triangle is set out with the tape having sides BC and CE of lengths equal to l and $\frac{l^2}{\text{R}}$ respectively. Point C thus set out is a point on the circle, and BC is a chord of the same length as AB. By producing BC its own length and constructing on it a triangle having the same sides as before another point will be found on the curve, and by continued application of the method any required length of curve may be set out.

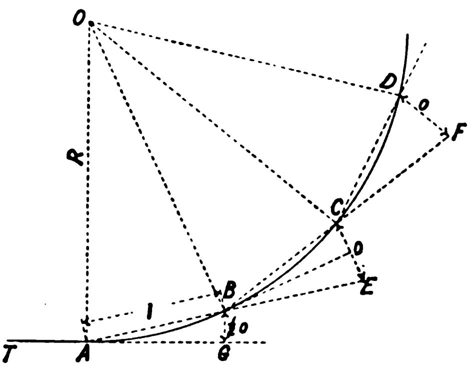

Fig. 238.—Offsets from Chords produced.

A tangent to the circle at point B would bisect the length CE so that the perpendicular offset to point C from the tangent line would have a length $\frac{l^2}{2\text{R}}$. Therefore, if TA is a tangent to the curve the first chord length AB will be set out by a *perpendicular* offset GB of length $\frac{l^2}{2\text{R}}$ set off from TA produced.

Example: A curve of 320 ft. radius is to be set out in 50-ft. chord lengths by the chord-offset method. Fig. 239 illustrates in detail the setting out of the first two points.

$$\text{Offset from chord} = \frac{l^2}{\text{R}} = \frac{50^2}{320} = 7{\cdot}81 \text{ ft.}$$

$$\text{Offset from tangent} = \frac{7{\cdot}81}{2} = 3{\cdot}90 \text{ ft.}$$

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To fix point B lay off in the first place a length of 50 ft. from A along the tangent line produced and erect an offset of 3·9 ft. This will give a point whose distance from A is rather more than 50 ft.

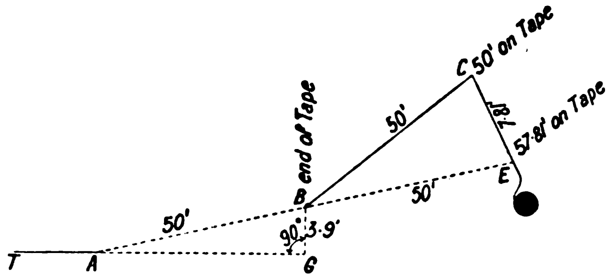

Fig. 239.—Setting out Curve by Chords and Offsets.

Adjust the point by moving it slightly parallel to the tangent line till the distance AB is 50 ft. and distance BG 3·9 ft. Point B is then accurately fixed. A distance of 50 ft. is then measured from point B along the line AB produced and point E is marked by an arrow or otherwise. Then to set out point C fix the zero end of the tape at B and hold the mark representing 57·81 ft. on the tape at point E. An arrow is then held inside the tape at the 50 mark, which is pulled out till both lengths are taut. The arrow is then at point C on the curve.

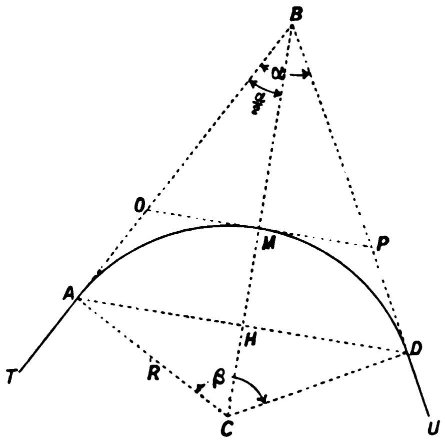

Fig. 240.—Curve Problems.

Curve Problems.—In Fig. 240 the two straight lines TA and DU are to be connected by a circular curve starting from point A. It is required to find the radius of the curve and to set out the curve by offsets from tangent lines.

If a theodolite is not available the problem may be solved as follows: the intersection point B of the lines TA and DU is first

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found. Distance BA to the tangent point is then measured and an equal distance BD laid off along the line BU. D will be the tangent point at the end of the curve. Length AD is measured and the half length AH becomes known.

Then in the right angled triangle AHB we know the lengths AB and AH, so that HB can be calculated. $HB = \sqrt{AB^2 - AH^2}$. Also if C is the centre of the circle the triangles CAB and AHB are similar, so that $\frac{CA}{AB} = \frac{AH}{HB}$, or $CA = R = \frac{AB \times AH}{HB}$.

$$\text{Distance BM} = \frac{R\,(AB - AH)}{AH}$$

$$\text{and distance AO or OM} = \frac{R^2\,(AB - AH)}{AB \times AH}.$$

Distances MP and PD are the same as distance AO, and by setting off the lengths AO and PD to the value found from the above formula the tangent line OP will be obtained. The curve may then be set out by offsets from the tangents AO, OP, and PD.

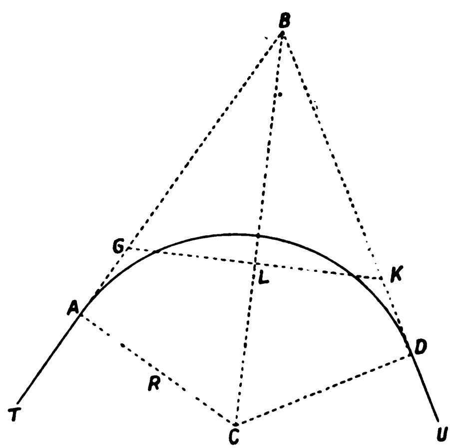

FIG. 241.—Curve Problems.

The converse of the above problem occurs when the radius of the curve is given. It then becomes necessary to find the positions of the tangent points. The intersection point B (Fig. 241) is found as before and two arbitrary equal lengths BG and BK are measured off along the tangent lines. Distance GK is measured so as to find the half length GL, and the distance BL is measured or calculated. Then A being the starting point of the curve, and C its centre, the triangles CAB and GLB are similar so that we have

$$\frac{AB}{CA} = \frac{BL}{LG}, \text{or } AB = \frac{CA \times BL}{GL} = \frac{R \times BL}{GL}.$$

By measuring off the length of AB as calculated by the above

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formula from the intersection point B along the tangent lines the starting and ending points A and D of the curve will be determined on the ground. The curve can then be set out by the methods already explained.

Trigonometrical methods may be employed for finding the lengths to the tangent points, &c., if the intersection angle is measured. In Fig. 240 the radius R is given and the intersection angle ABD has been measured and is equal to a so that the angle $ABC = \frac{a}{2}$

Then $AB = CA \cot \frac{a}{2} = R \cot \frac{a}{2}$.

Also $CB = \frac{R}{\sin \frac{a}{2}}$. But $BM = CB - CM = CB - R$.

$$\text{Therefore } BM = \frac{R}{\sin \frac{a}{2}} - R = R \left(\frac{1}{\sin \frac{a}{2}} - 1\right) = R\left(\frac{1 - \sin \frac{a}{2}}{\sin \frac{a}{2}}\right)$$

$$\text{Also } AO \text{ or } OM = BM \tan \frac{a}{2} = R\left(\frac{1 - \sin \frac{a}{2}}{\cos \frac{a}{2}}\right).$$

To find the length of the curve AMD.

Curve subtends angle β at centre of circle, where $\beta = 180 - a$.

Length of curve $L = R\beta$ when β is stated in radians.

$= \frac{R\beta}{57 \cdot 30}$ " " " degrees.

$= \frac{R\beta}{3438}$ " " " minutes.

Setting out Curves with Theodolite.—The setting out of circular curves with the theodolite depends on the geometrical principle that a given length of arc subtends the same angle at any point of the circumference. Thus in Fig. 242 if A1, 12, 23 are equal arcs the angles 3C2, 3A2, 2A1, 1AB are equal to each other and equal to half the angle 3O2 which the arc subtends at the centre of the circle. If l is the length of the arc and R the length of the radius the angle subtended at the centre of the circle has been shown to be equal to

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$\frac{l}{R} \times 3{,}438$ minutes. The angle δ subtended by an arc l at the circumference is therefore equal to $\frac{l}{R} \times 1{,}719$ minutes. This is known as the deflection angle for the curve because a theodolite set up at the tangent point A will require to be deflected to the right from the tangent line through successive values of the angle in order to sight towards the points, 1, 2, 3, &c. The first thing to be done, therefore, preparatory to setting out a curve by theodolite is to calculate, or find from tables prepared for the purpose, the value of the single deflection angle δ, and the values of 2δ, 3δ, 4δ, &c., up to $n\delta$, where n is the number of points to be set out at one planting of the instrument. In practice the curve is set out in chord lengths instead of arcs, but the lengths are made so short that the error introduced is negligible. Commonly accepted limits are, when working in chains of 66 ft., to set out the points at whole chain intervals when the curve is over twenty chains radius and at half-chain intervals when the radius is under twenty chains, and when working in feet to set out the points 100 or 50 ft. apart according as the radius is over or under 2,000 ft. To set out the first point 1 the theodolite is planted at A and sighted along the tangent line in the direction AB with the vernier set to zero. The single deflection angle δ is then turned off to the right so that the theodolite points in the direction A1. At the same time one end of the chain is held at A, an arrow is held on the chain at length l from A (this is usually at the other end of the chain), and the chain is straightened out and swung about A as a centre until the arrow is brought exactly into the line of sight of the theodolite. The arrow is then at point 1 on the curve and may be fixed in the ground, or the point may be marked permanently by driving in a peg. To set out point 2 the

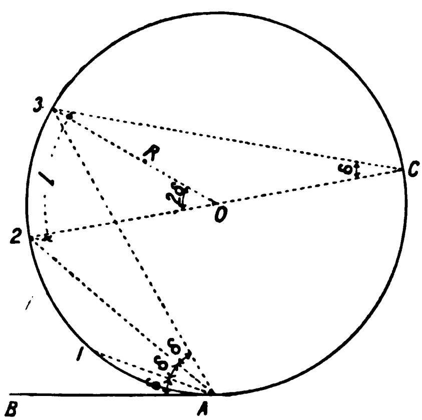

FIG. 242.—Setting out Curve by Theodolite.

theodolite is further turned to the right through a deflection angle so that the reading on the circle becomes equal to 2δ. It then looks towards point 2. The end of the chain is now held at point 1 and a length l of the chain is taken and swung about this point till the arrow is in the line of sight of the theodolite. The arrow then marks point 2. Further points are set out in succession round the curve by turning the theodolite through a deflection angle each time and measuring a distance l from the point last fixed.

When in proceeding with the setting out the points become inconveniently far away from the instrument or when the view becomes obstructed, and in any case before the sum of the deflection

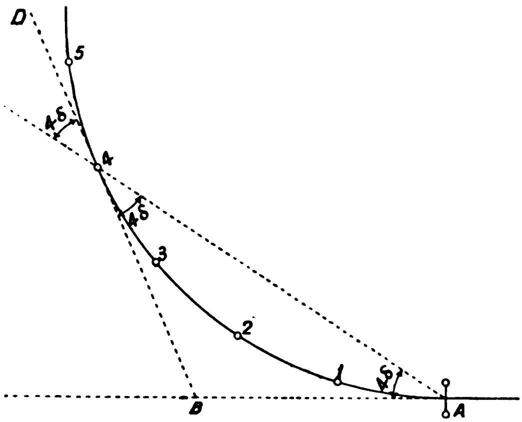

FIG. 243.—Changing Position of Theodolite.

angles reaches 45°, the theodolite must be shifted forward to the point last set out and a fresh start made from a new tangent line, as shown in Fig. 243. The curve has been set out, let us suppose, up to point 4 with the theodolite planted at point A and the conditions render it necessary that the theodolite should be shifted forward in order to continue the setting out. It is planted at 4, the vernier is set back to zero, the telescope is sighted back on point A and the lower plate is clamped. The telescope is then transited so as to point forward towards C along the line A4 produced. The vernier reading is still zero. The telescope is now turned to the right through an angle equal to the angle used in setting out point 4, that is in this case through an angle equal to 4δ, and points towards D. An inspection of the figure will show

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that the telescope is now looking along the line of the tangent at point 4. Point 5 will, therefore, be set out by turning the telescope further to the right through the deflection angle δ making the reading on the circle equal to 5δ or the same as it would have been if point 5 had been set out from A. Points 6, 7, &c., are set out by turning the telescope so that the circle readings are equal to 6δ, 7δ, &c.

Instead of setting the vernier to read zero when sighting back from the turning peg (peg 4) to the tangent point, it is often more convenient to set it to read an angle to the left of zero equal to the whole deflection angle of the turning point. In this case we would sight on A with the vernier reading the angle $360°—4\delta$, so that when the telescope is transited and turned to the right through the angle 4δ the reading will be zero and the telescope will be pointing along the tangent. Points 5, 6, 7, &c., will then be lined in by again setting off the deflection angles δ, 2δ, 3δ, &c., to the right from point 4, exactly as from point A.

Example. The centre line of a portion of railway is being set out and marked by pegs driven in at every chain length of 100 ft. A curve of 2,500 ft. radius occurs connecting two straights. We will follow through the operations and calculations required in setting out the curve.

The first operation is to find the intersection point of the straight lines. The lines are ranged out roughly by eye in the first place so as to find the approximate position of the point. Then with the theodolite planted on one of the straights and looking along the line towards the intersection point two arrows are ranged into line a few yards apart and so as to occur one on either side of the required point. A string is stretched between the arrows. The theodolite is then planted on the other straight, the telescope is sighted into line and pointed towards the intersection point. An arrow is then set at the point where the line of sight cuts the string and marks the intersection of the straights. A peg may be driven as a more permanent mark.

The theodolite is now set up over the intersection point and the angle between the straights is measured. Assume that this is 148° 20′. The distance from intersection point to tangent point can now be calculated and must be found to enable the tangent points to be set out.

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Half intersection angle $= 74° \ 10'$.

Distance to tangent point $= \text{R} \cot \frac{a}{2} = 2{,}500 \cot 74° \ 10'$.

$$\begin{aligned} \text{Log } 2{,}500 &= 3\cdot39794 \\ \text{Log cot } 74° \ 10' &= 9\cdot45271 \\ \hline & \ \ 2\cdot85065 = \log 709\cdot0. \end{aligned}$$

The distance of 709 ft. is, therefore, measured backwards along each straight from the intersection point in order to fix the starting and finishing points of the curve.

On pegging out the straight up to the commencing point of the curve it will be a mere coincidence if a peg occurs at the tangent point. Let us assume that the distance from the last peg on the straight, say, peg No. 71, to the tangent point measures 39 ft., so

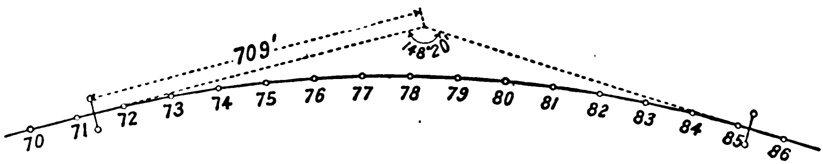

FIG. 244.—Setting out Curve.

that the distance from the tangent point to the first peg on the curve, peg No. 72, will be 61 ft.

$$\text{Deflection angle for 61 ft.} = \frac{61}{2{,}500} \times 1719 = 42 \text{ minutes.}$$

$$\text{Deflection angle for 100 ft.} = \frac{100}{2{,}500} \times 1719 = 68\cdot76 \text{ minutes.}$$

Angle which arc of 100 ft. subtends at centre of circle $= 68\cdot76 \times 2 = 137\cdot52$ minutes.

This latter figure may be used in finding the whole length of the curve. The angle which the whole curve subtends at the centre of the circle is equal to 180° *minus* the intersection angle. In this case the curve subtends an angle of $180° - 148° \ 20' = 31° \ 40' =$ 1,900 minutes. Then length of curve $= 100 \times \frac{1900}{137\cdot52} = 1381\cdot6$ ft.

The distance from starting point to first peg on curve is 61 ft., leaving a remainder of 1320·6 ft. Therefore, following on the first peg 13 complete arcs of 100 ft. will require to be set out, and there

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will be a closing length of 20·6 ft. to the end of the curve. See Fig. 244, which shows the curve with the positions of the pegs plotted to scale.

The following is a list of the angles required in setting out the points. The angle to set out the first point, peg No. 72, is the deflection angle for 61 ft., namely, forty-two minutes. It is very desirable to get rid of the occurrence of this odd amount in the series of deflection angles, and this is effected by commencing with the theodolite set to read forty-two minutes to the left of zero, that is, the angle 359° 18′, so that when the theodolite is turned from the tangent to the right through 42′, it will be reading 0° 0′ and pointing to peg 72. The deflection angles for the first ten pegs for a wide range of curves can be obtained without calculation from various sets of curve tables, such as Kröhnke's.

Angles for Setting out Curve of 2,500 ft. Radius in Arcs of 100 ft.

Tangent	359° 18′	Peg No. 77	5° 43¾′
Peg No. 72	0° 0′	,, 78	6° 52½′
,, 73	1° 8¾′	,, 79	8° 01¼′
,, 74	2° 17½′	,, 80	9° 10′
,, 75	3° 26¼′	,, 81	10° 18¾′
,, 76	4° 35′	,, 82	11° 27½′

The theodolite being planted at the start of the curve and sighted along the tangent line with the vernier set to read 359° 18′, peg No. 72 is set out by turning the theodolite to the right through 42′ so as to read 0° 0′ and lining in the peg at a distance of 61 ft. from the tangent point. The succeeding pegs are set out in turn by the deflection angles shown in the table and at distances of 100 ft. from each other. Even although the view is unobstructed right round the curve it would be advisable to shift the theodolite once, say, after peg No. 79 has been fixed. Having removed the instrument and planted over peg No. 79 set the vernier to read to the left of zero an amount equal to the whole deflection angle already turned through. In this case the whole deflection angle up to peg 79 is 8° 01¼′ + 0° 42′ = 8° 43¼′, and we set the vernier to read 360°—8° 43¼′ = 351° 16¾′ and sight back to the tangent point.

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The telescope is then transited and turned to the right to read 0° 0′ when it points along the tangent at peg 79. The succeeding pegs will then be lined in by again laying off the same series of deflection angles, that is, peg 80 is given by the angle 1° 8¾′, peg 81 by the angle 2° 17½′, and so on. The accuracy of the work is tested at the closing length from peg No. 85 to the end tangent point. This distance should measure the calculated amount, namely, 20·6 ft., and the angle read when the theodolite is sighted on to the tangent point, added to the total deflection angle up to peg 79, should be equal to half the angle subtended by the curve at its centre, in this case half of 31° 40′ or 15° 50′.

Curve to Left.—The foregoing descriptions of the setting out of curves by theodolite apply to curves which turn off to the right hand from the tangent line. A slight difference in the procedure arises in setting out a curve to the left hand due to the fact that theodolites in this country are usually graduated only in right-hand or clockwise direction. If, therefore, in setting out a curve to the left we start with the telescope looking along the tangent line and the circle reading at zero, the angle for setting out the first point will be got by subtracting the deflection angle from 360°. The angle for the second point will be obtained by subtracting the deflection angle from the angle for the first point and so on. The subtraction of degrees, minutes, and fractions is an awkward process and liable to be a fruitful source of arithmetical error, and should, therefore, be avoided if possible. This may be done by proceeding as shown in Fig. 245 and accompanying table, which give a comparison of the angles required for setting out the same curve to the right hand and to the left hand. The angles are calculated for a sufficient number of points as for a curve to the right. Then to set out a left-hand curve these angles are taken in the reverse order. In the case illustrated the theodolite is sighted along the tangent line with the circle reading set to 9° 35′. Then peg No. 1 to the left is set out by the angle 8° 37½′, peg No. 2 by the angle 7° 40′, and so on.

Inaccessible Intersection Point.—Referring to Fig. 241, if the intersection point B is inaccessible it may be possible to run and measure some line such as GK between the tangent lines and to measure the angles AGK and GKD. The lengths of the sides of

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Angles for Curve to Left.	Angles for Curve to Right.
Tangent 9° 35′	Tangent 0° 0′
Peg No. 1 = 8° 37½′	Peg No. 1 = 0° 57½′
„ 2 = 7° 40′	„ 2 = 1° 55′
„ 3 = 6° 42½′	„ 3 = 2° 52½′
„ 4 = 5° 45′	„ 4 = 3° 50′
„ 5 = 4° 47½′	„ 5 = 4° 47½′
„ 6 = 3° 50′	„ 6 = 5° 45′
„ 7 = 2° 52½′	„ 7 = 6° 42½′
„ 8 = 1° 55′	„ 8 = 7° 40′
„ 9 = 0° 57½′	„ 9 = 8° 37½′
„ 10 = 0° 0′	„ 10 = 9° 35′

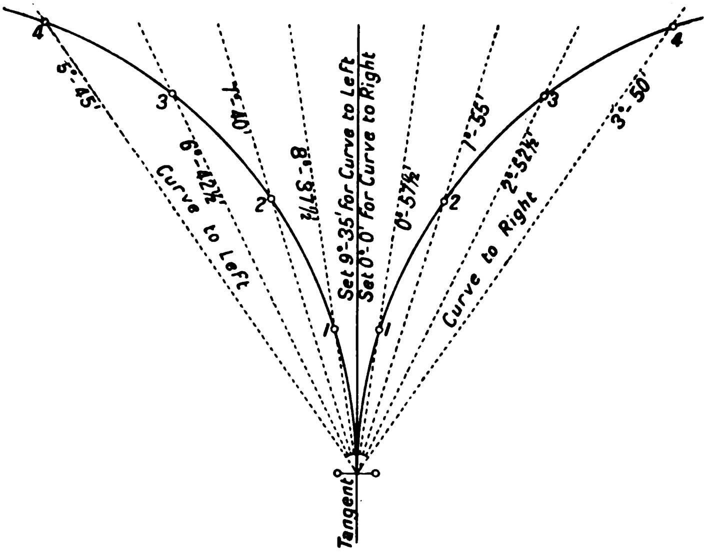

FIG. 245.—Curves to Right and Left.

the triangle BGK and the tangent lengths BA and BD could then be calculated.

$$\text{Angle BGK} = 180° - \text{AGK}$$
$$\text{,, BKG} = 180° - \text{GKD}$$
$$\text{and intersection angle GBK} = 180° - (\text{BGK} + \text{BKG}).$$

In the triangle BGK the base GK and the angles at G and K are, therefore, known, and the lengths BG and BK can be calculated by

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the method given on p. 188. Also, since the intersection angle at B is known, the tangent lengths BA and BD can be calculated by the method given on p. 301. The distances GA and KD are then found by subtraction, and when measured off backwards from points G and K serve to fix the tangent points on the ground. The setting out of the curve can then proceed in the usual manner.

Obstructions in Setting out Curves.—Any point on a curve can be set out independently of any other by laying off its whole deflection angle and measuring the chord length along the line so given. If the whole deflection angle is Δ the chord length is $= 2\ R \sin \Delta$, R being the radius of the curve. This points to a method of passing obstructions which occur on the line of the curve, or on the lines of sight of the theodolite. The setting out of the points affected by the obstruction is omitted and the next point beyond the obstruction is set out by the whole deflection angle and chord length. The curve is then continued in the usual manner, and the omitted portion may be dealt with when the obstacle is removed.

Many of the obstacles in the nature of obstructions to the line of view of the theodolite can be easily overcome by suitably choosing the points at which to shift the instrument.

Where the line of the curve is much obstructed the difficulty may often be overcome by running a parallel curve of smaller or larger radius. The actual curve is then set out by radial offsets as the obstructions are removed. If it is desired, in the case of a parallel curve, to keep the correct centre line chainage the interval between the points must be altered in proportion to the increase or reduction of the radius, but the deflection angles will remain the same. The setting out of a parallel curve has the advantage that the pegs may remain and be of use during construction of the works, whereas centre-line pegs are ordinarily lost as soon as construction commences.

Setting out Building Work.—The principal operations required in the setting out of building work are the laying off on the ground of right angles and other angles with the theodolite or otherwise; the setting out of straight and curved lines; and the accurate measuring and marking-off of distances. Usually only the principal building lines are required to be set out by the engineer or architect. Wooden pegs or stakes are driven into the ground to

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mark the lines and points, accurate points being defined by nails driven into the heads of the pegs.

Methods of setting out angles, ranging lines, &c., with and without the theodolite, are given in Chapters VII. and XV. In the measuring of distances much greater precision is required than in ordinary surveying. In important work, such as foundations to carry a steelwork superstructure, the essential dimensions should be set out accurately to within $\frac{1}{4}$ in. On rough or sloping ground and in transferring lines, from the surface of the ground to the bottom of foundation trenches, &c., very careful use of the plumb-bob is necessary if accurate work is to be accomplished.

An example of the setting out of the principal lines of building work is illustrated in Fig. 246, which shows in plan and elevation a bridge carrying a road over a railway. The principal building lines are the face lines of abutments and wing walls at the level of the top of the concrete foundation, and it is these which would ordinarily be pegged out. Before the bridge can be set out the engineer must know the chainage to the centre of the bridge (point A), the angle of skew, the clear span and clear width between parapets, and must have a drawing showing the masonry in plan. Dimensions of wing walls, &c., may be marked on the drawing, or may require to be scaled.

The skew distances AE, ME and EK will not usually be marked on the drawing, but are required in the setting out. They might be scaled from an accurate drawing, but should be calculated. The corresponding square distances are 13 ft. 6 ins., 17 ft. 6 ins., and 19 ft. 6 ins. The skew distance in each case is obtained by multiplying the square distance by the cosecant of the angle of skew (65° 10′), and the results are 14 ft. 10$\frac{1}{2}$ ins., 19 ft. 3$\frac{3}{8}$ ins., and 21 ft. 5$\frac{7}{8}$ ins. respectively. The first operation in the setting out is to put in the centre peg at A. On a straight centre line of railway already staked out the theodolite would be set up on an adjacent centre-line peg, sighted on to another centre-line peg at some distance away, and then tilted down to line in the peg at A, whose position is fixed by measurement from the nearest centre-line peg. The exact point is marked by a nail. The theodolite is then set up over peg A and centred over the nail. With the vernier set to zero the telescope is sighted on a centre-line peg B at some distance away, and an angle of 65° 10′ is then turned off to the right. The

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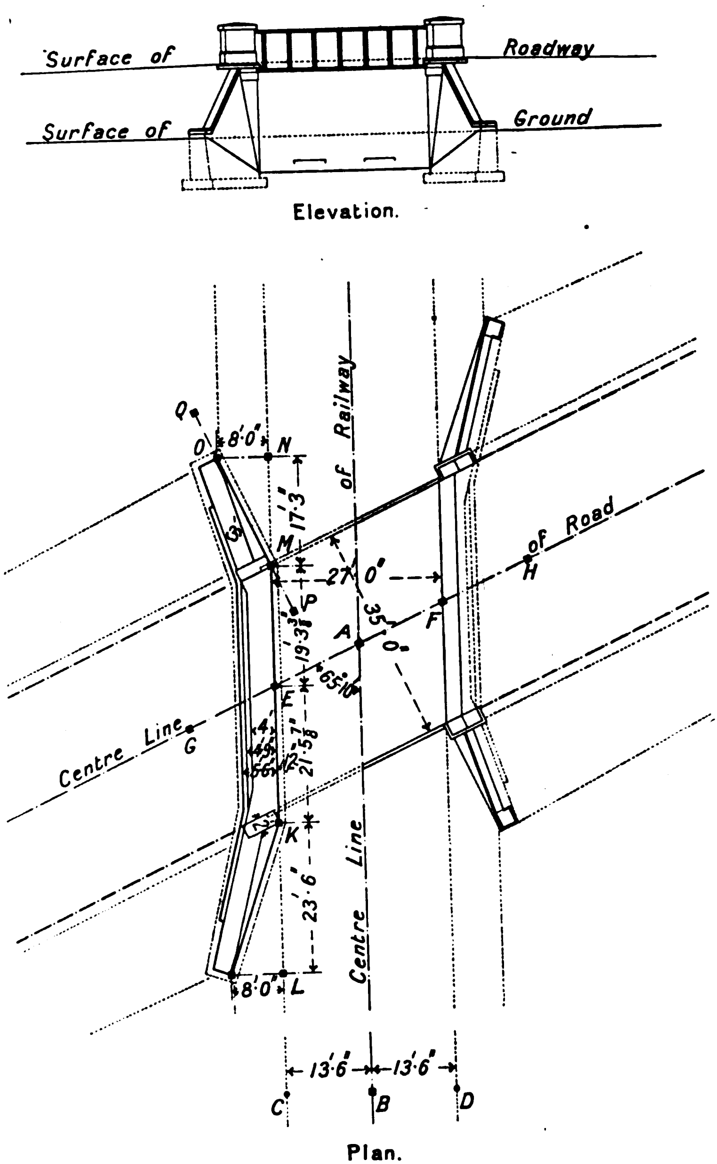

FIG. 246.—Setting out a Bridge.

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telescope now gives the centre line of the road and peg E is lined in at 14 ft. 10½ ins. from A. Transfer peg G is also lined in clear of the site of the excavation, and with the telescope transited corresponding pegs F and H are put in on the other side of A. The theodolite is then set up over E and sighted on a mark C set out at 13 ft. 6 ins. from B square to the centre line of railway. This gives a line of sight parallel to the centre line of railway, and points K and L may be lined in at the distances marked, and, by transiting the telescope, points M and N may also be lined in. The corners of the wing walls are fixed by square offsets of 8 ft. from pegs N and L. The face line of the other abutment would be set out in a similar manner, with the theodolite set over peg F. When the building-line pegs O, M, E, K, &c., have been put in, the outline of the concrete foundation may be marked out on the ground in accordance with figured or scaled dimensions obtained from the drawing, working from the building line as a base line. When excavation commences, however, these pegs will be lost, and it is necessary to have such transfer pegs as will enable the building lines to be recovered at any time, and particularly when building is about to commence on top of the concrete foundation. A line stretched over pegs G, A and H will give the centre line of the road at any time, and points E and F on that line can be recovered by measuring the proper distance from A, and can be transferred to the surface of the concrete by plumbing. A line stretched from L to N will give the face line of the abutment, and points M and K can be obtained by measurement from E, as in setting out. It would be advisable, however, to have transfer pegs for each wing wall, such as P and Q, set out at a round number of feet from the corners and so as to be clear of the excavation. These preserve the line of the wing wall and enable points O and M to be readily recovered and transferred down to the foundations.

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CHAPTER XX

CALCULATION OF AREAS

In this chapter consideration is given to the methods in common use for computing areas from a survey plan, such as by dividing the area into triangles, parallel strips, squares, &c., also the determination of areas from offsets by the trapezoidal and Simpson's rules, and the finding of areas by the planimeter. Methods of computing areas of plots and areas of traverses directly from the field measurements are dealt with, and the effect of shrinkage of plans and the allowance to be made for shrinkage in taking out areas are considered.

Square Measure.—The British units of area employed in surveying are as given in the following statement :—

1 sq. ft. = 144 sq. ins.
1 sq. yard = 9 sq. ft. = 1,296 sq. ins.
1 sq. pole = $30\frac{1}{4}$ sq. yards = $272\frac{1}{4}$ sq. ft.
1 sq. chain = 484 sq. yards.
1 rood = 40 sq. poles = 1,210 sq. yards.
1 acre = 4 roods = 10 sq. chains = 4,840 sq. yards = 43,560 sq. ft.

The following figures give the relationships existing between some of the metrical and some of the British units of area :—

1 sq. metre = 10·7639 sq. ft. = 1·1960 sq. yards.
1 hectare = 10,000 sq. metres = 2·4710 acres.
1 sq. kilometer = 100 hectares = 0·3861 sq. miles.
1 sq. ft. = 0·0929 sq. metres.
1 sq. yard = 0·8361 sq. metres.
1 acre = 0·4047 hectares.

For small areas, such as building lots, the unit of area commonly used is the square yard, occasionally the square foot. For large areas the acre is almost universally employed. The method of

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expressing fractions of an acre in roods, poles, &c., is very cumbersome. It is much simpler to express the fraction decimally.

The following table gives formulæ for the areas of the geometrical figures which are most commonly of use in the calculation of survey areas :—

Areas of Geometrical Figures.

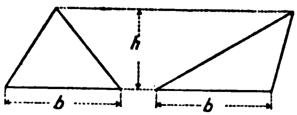
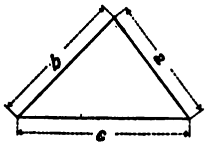
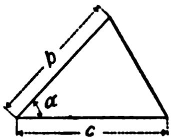
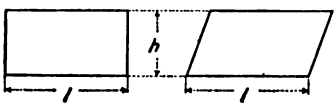
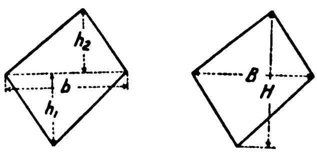
Figure.	Area.
Triangles. 	Area $= \frac{1}{2} bh$.
	Area $= \sqrt{s(s-a)(s-b)(s-c)}$ where $s = \frac{a+b+c}{2}$
	Area $= \frac{1}{2} bc \sin a$.
Parallelograms. 	Area $= lh$.
Quadrilateral. 	Area $= \frac{1}{2}b(h_1 + h_2)$ or $= \frac{1}{2}BH$.

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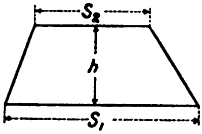
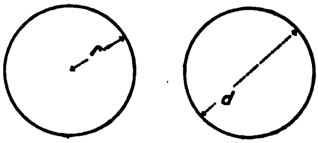
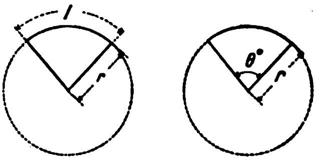
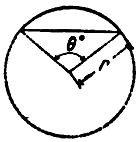
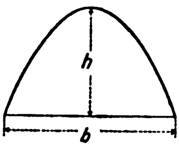
Figure.	Area.
Trapezium. 	Area $= \frac{1}{2}h(s_1 + s_2)$.
Circle. 	Area $= 3{\cdot}1416\, r^2$ or $= 0{\cdot}7854\, d^2$.
Sector of Circle. 	Area $= \frac{1}{2}\, lr$ or $= \dfrac{3{\cdot}1416\, r^2\, \theta}{360}$ $= 0{\cdot}008727\, r^2\, \theta$, θ being given in degrees.
Segment of Circle. 	Area $=$ $r^2\,(0{\cdot}008727\theta° - \frac{1}{2}\sin\theta)$, θ being given in degrees.
Flat Segment of Circle. 	Area $= \dfrac{2}{3}\, lh$ approx.
Parabola. 	Area $= \dfrac{2}{3}\, bh$.

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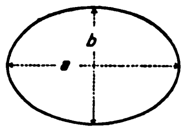
Figure.	Area.
Ellipse. 	Area $= 0{\cdot}7854\, ab$.

Area of Land.—For surveying purposes the distance between two points is taken as the straight distance between the projections of the points on a horizontal plane. Similarly, the area of a plot of land is not taken as the whole surface exposed following the undulations and irregularities, but is taken as the area contained within a projection of the boundaries on a horizontal plane. Areas measured from an accurate survey plan will fulfil this condition.

Methods of taking out Areas.—There are two general methods of obtaining areas :—

(*a*) By scaling or measuring from a survey plan.

(*b*) By direct calculation from field measurements.

Areas are most commonly obtained by scaling from a plan. Greater accuracy can be obtained by method (*b*), but this method is in general only applicable when the measurements have been specially taken and arranged with the view to computation of areas.

Areas from Survey Plan.—(1) By dividing the area up into geometrical figures. The plot whose area is required may be divided up into geometrical figures of the forms shown in the foregoing table. Its area will be obtained by adding together the areas of the separate figures.

Any straight-sided figure may conveniently be divided up into triangles. The polygon ABCDEF (Fig. 247) may, in order to calculate the area, be divided into the four triangles shown. The triangles are taken in pairs. AC is scaled as the common base of the triangles ABC and AFC, and FD is also scaled as the common base of two triangles. The heights h_1, h_2, h_3, and h_4 are also scaled off. The area of the figure is then equal to $\frac{1}{2}$ AC $(h_1 + h_2) + \frac{1}{2}$ FD $(h_3 + h_4)$.

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Any straight-sided figure may, by simple geometrical construction, be reduced to a single triangle of equal area, and this furnishes a useful and rapid method of taking out areas. It is required to construct a single triangle equal in area to the quadrilateral ABCD (Fig. 248). Divide the figure up into two triangles by the diagonal BD. Through C draw a parallel to BD to meet AD produced in E. Then if line BE be drawn in, the triangles BCD and BED will be of equal area since they have the same base BD and their perpendicular heights from base to apex are the same. The area of the quadrilateral ABCD formed of the two triangles ABD and DBC is, therefore, equal to the area of the triangle ABE formed of the two triangles ABD and BED. By continued application of the above method any figure having a number of straight sides may be reduced to a single triangle. The constructions to obtain a single triangle equivalent in area to the six-sided figure ABCDEF are illustrated in Fig. 249. Eg is drawn parallel to DF. This gives triangle DgF equivalent to triangle DEF. Bh is drawn parallel to CA, giving triangle ChA equivalent to triangle CBA. Finally, Ck is drawn parallel to Dh, giving triangle Dkh equal to the triangle DCh. The whole triangle kDg is then equal in area to the figure ABCDEF.

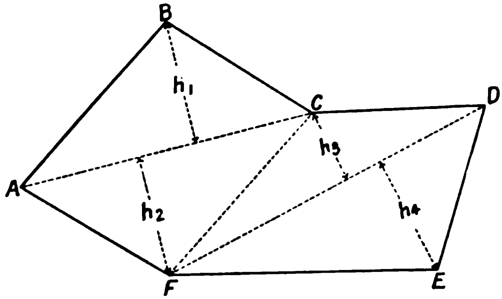

FIG. 247.—Area of Figure by Subdivision.

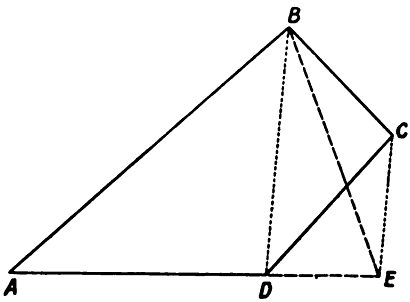

FIG. 248.—Single Triangle equal to Quadrilateral.

The foregoing method of reducing a figure to a single triangle is rapid, as only a few of the construction lines shown on the diagram, or portions of them, need actually be drawn. Thus to obtain point g the edge of the parallel ruler (or set-square) would be laid across the points F and D and the ruler would then be rolled till the edge passed through E. A short stroke drawn across the line AF produced would then fix point g.

The degree of accuracy with which the area of the single triangle corresponds to the area of the figure will depend on the draughtsmanship. The benefit of the method lies in the greatly reduced amount of scaling and calculation involved in finding the area of a single triangle as compared with several separate triangles, and the correspondingly reduced liability to error. The method may often be employed to give an independent check on an area calculated in some other way, and *vice versâ*.

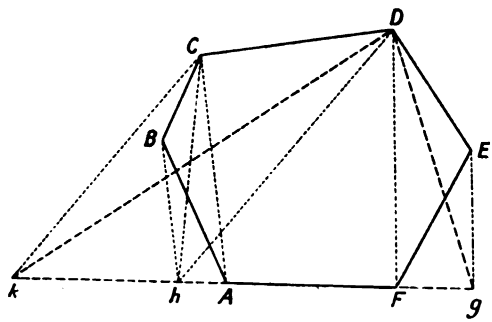

FIG. 249.—Area by Single Equivalent Triangle.

Methods of dealing with irregular and curved boundaries are illustrated in Fig. 250. Irregular boundaries are "equalised" by replacing them with straight sides arranged so as to include and exclude equal small portions of the area as nearly as may be judged by the eye, the purpose being to obtain a straight-sided figure of exactly the same area as that contained within the irregular boundary. The straight-sided figure would be dealt with by the method of triangles.

The irregular sides AB and ED (Fig. 250) are equalised by the dotted lines shown.

Regular curved boundaries, such as BCD in the figure, are generally dealt with by dividing up into flat segments, such as BC and CD, the area of these being calculated by the formula for flat segments. Thus the whole area of the above figure would be equal to

$$\tfrac{1}{2}\,\mathrm{BE}\,(h_1 + h_2) + \tfrac{1}{2}\,\mathrm{BD}\,h_3 + \tfrac{2}{3}\,\mathrm{BC}\,h_4 + \tfrac{2}{3}\,\mathrm{CD}\,h_5.$$

(2) By dividing the area up into parallel strips of equal width. The usual method of doing this is to draw parallel lines on a piece of

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tracing cloth or paper and to lay this over the area, thus dividing it into strips. The area contained in any strip will be equal to the mean length between boundaries multiplied by the width of the strip, and the whole area will be equal to the sum of the lengths of all strips multiplied by the width of a strip. The width of strip should be such as will render the calculation of the area easy. If the area is desired in actual square inches of paper the width of strip would be made either 1 in. or some convenient simple fraction or multiple of an inch. If the area is to be taken from a plan and given in acres the width of strip would be taken as equal to one chain to the scale of the plan or some suitable fraction or multiple

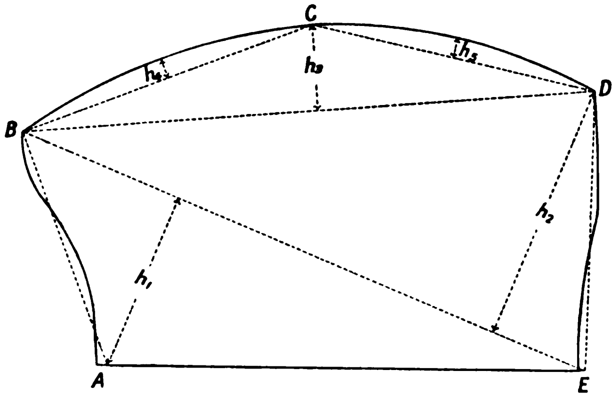

FIG. 250.—Area within Irregular Boundaries.

of a chain, because of the simple relationship which exists between square chains and acres, namely one acre = ten square chains. The width of strip should also be limited to such a size as will cause the portions of boundary intercepted between adjacent lines to be sensibly straight, so that the mean length of a strip may be taken as the length along its middle line.

The method is illustrated in Fig. 251, where ABCD is the figure whose area has to be determined, and the dotted lines represent the parallel lines drawn on a superposed piece of tracing cloth or paper. The whole area with the exception of an irregular portion at each end is comprised within parallel strips. The areas of these end portions require to be separately calculated. The area of the remainder is equal to $d\,(l_1 + l_2 + \cdots + l_n)$, where d is the width of

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strip and l_1, l_2, &c., are the mean lengths of successive strips. There are several devices for rapidly summing up the lengths of strips. Where the lengths are short they may be summed up with a pair of dividers, the points being set first to the length of the first strip, and then further opened out by the length of the second strip, and so on. Another simple method is to take a long strip of paper and mark off the lengths successively along this. The total length obtained may be scaled in one operation. An improvement on the above method consists in summing up the lengths by means of a scale with a sliding index. The graduations of the scale may be arranged so that the reading of the index gives the area in acres. The appliance is then known as a "computing scale."

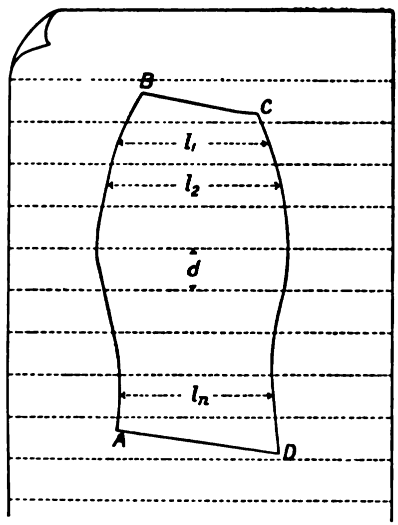

FIG. 251.—Area by Parallel Strips.

(3) By dividing the area up into squares. This method is not essentially different from the preceding. Parallel lines are drawn on tracing cloth or paper in two directions at right angles to each other so as to form squares of a convenient fraction or multiple of the unit area employed. The tracing is laid down and fixed over the figure whose area is to be ascertained. The area comprised within the boundary of the figure will consist of a certain number of whole squares together with a number of fractions of squares of triangular and trapezoidal form around the boundary. The number of whole squares is counted, and the areas of the separate fractions are separately calculated.

The method of finding an area by dividing it into squares is generally less convenient than the method by parallel strips.

(4) Areas by means of offsets. The method of taking out areas by offsets is most commonly applicable where the area is in the form of a narrow strip which continues in one general direction for

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some distance. In dealing with plan areas the offset method may be adopted for irregular boundaries instead of the equalising method already explained. The external sides of the triangles or other shapes into which the figure is divided are then kept entirely within the boundary, and the excluded portions of area are dealt with by the offset method. The offset method is also adapted to finding areas directly from measurements taken on the ground.

The offsets may be taken at equal or unequal intervals apart. The usual case of offsets taken at equal intervals from a straight base line to a curving boundary is illustrated in Figs. 252 and 253.

The common distance between offsets is d, the lengths of the end

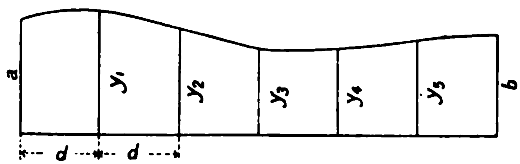

FIG. 252.

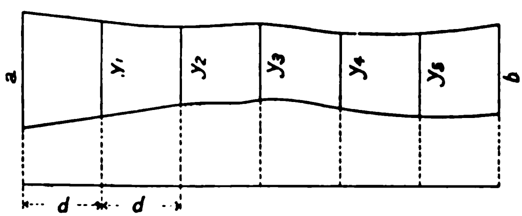

FIG. 253.
Areas by Offsets.

offsets are a and b, and the lengths of the intermediate offsets are successively y_1, y_2, y_3, &c. If the boundary is straight between successive offsets the total area will be equal to $d\left(\frac{a}{2} + y_1 + y_2 + y_3 +, \text{\&c.}, + \frac{b}{2}\right)$ or otherwise equal to $d\left(\frac{a+b}{2} + \Sigma y\right)$ (trapezoidal rule), where a and b are the lengths of the end offsets or ordinates and Σy is the sum of all the intermediate ordinates.

A slight variation of the above method consists in measuring the offsets at the middle of each equal space, as shown in Figs. 254 and 255, instead of at the ends of the spaces. The area is then equal to $d\Sigma y$ (mean ordinate rule).

Where the boundaries are regularly curving the area will be more

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accurately determined by means of Simpson's rule than by either of the two foregoing rules. To apply Simpson's rule the ordinates are taken as in Figs. 252 and 253, and there must be an even number of equal spaces. Referring to these figures the rule is:—

$$\text{Area} = \frac{d}{3}(a + 4y_1 + 2y_2 + 4y_3 + 2y_4 + 4y_5 + b).$$

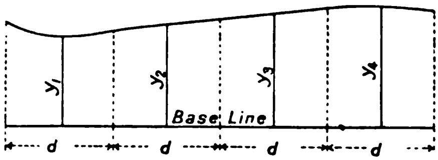

FIG. 254.

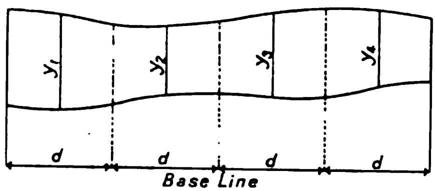

FIG. 255.
Areas by Mean Ordinates.

Where, as in Fig. 256, the boundary is a series of straights of varying lengths, or where it can be equalised into such a form, the obviously correct method of taking out the area is by offsets taken to the angles of the boundary. The whole area will be obtained by adding together the areas of the separate trapezoids into which it is divided by the offsets. Thus the area of Fig. 256 is equal to $\frac{1}{2}d_1(y_1 + y_2) + \frac{1}{2}d_2(y_2 + y_3) + \frac{1}{2}d_3(y_3 + y_4) + \frac{1}{2}d_4(y_4 + y_5)$.

Stated in a more convenient form for calculation, the area is equal to $\frac{1}{2}(y_1s_1 + y_2s_2 + y_3s_3 + y_4s_4 + y_5s_5)$, where s_2 is the sum of the two spaces adjacent to the ordinate y_2, and so on, as shown in the figure. In taking out an area from a survey plan by this method, the distances s_1, s_2, s_3, &c., would be scaled off complete. They would not be economically obtained by scaling the separate spaces d_1, d_2, d_3, &c., and adding them together.

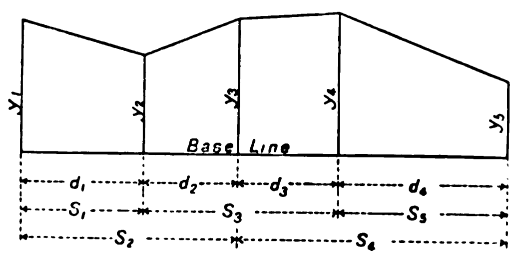

FIG. 256.—Area by Offsets.

(5) Areas by Planimeter. There are various forms of planimeter or mechanical instrument for the measurement of areas. In most forms the operation of finding an area consists in guiding the tracing

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point of the instrument once completely round the boundary of the figure and then taking the reading of an index on a scale. The areas of figures of irregular outline are much more easily obtained by planimeter than by any other means.

Amsler's polar planimeter will be described, as it is in very common use for measuring areas. The essential elements of the instrument are shown diagrammatically in Figs. 257 and 258, which illustrate the two forms in which it is constructed. In each case there are two bars hinged together. One bar is fitted with a tracing point at one end and has a freely revolving roller with a graduated scale reading against a fixed index near the other end. The other bar has at its extremity a sharp point for fixing into the paper. In the form shown in Fig. 257 the tracing point D and the roller B are on opposite sides of the hinge C, while in the form of Fig. 258

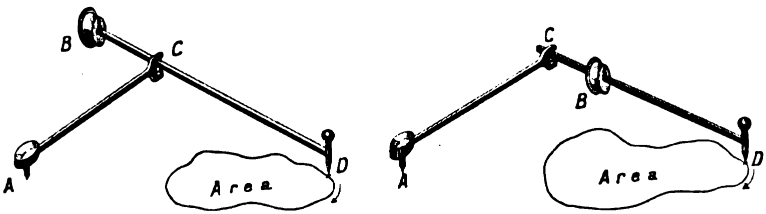

FIG. 257. FIG. 258.

Elements of Planimeter.

the tracing point and the roller are on the same side of the hinge. The instrument is fixed to the paper in each case at the point A and rests on the roller B and tracing point D, which slide over the paper during the operation of measuring an area.

In using the planimeter the tracing point is set to a definite mark on the boundary, the roller scale is set to zero, and the tracing point is then moved in a clockwise direction round the boundary and back to the starting point. The whole instrument swings meantime about the point A and the motion of point B will be partly by sliding and partly by rolling. While the tracing point is making the circuit the roller will revolve a certain amount forwards and a certain amount backwards, but for a complete circuit of an area there will always be a balance of rotation in one direction. The amount of this rotation is proportional to the area of the figure, and is read off on the scale.

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The actual construction of two forms of Amsler's planimeter is shown in Figs. 259 and 260. In Fig. 260 the hinge is between the roller and the tracing point and the hinge and roller are at a fixed distance apart, being attached to a frame which slides on the bar carrying the tracing point. The sliding frame carries also the vernier and counter for recording the rotation of the roller. Areas can be read off directly in the required units to various definite scales by altering the distance from the tracing point to the hinge. The required distances for the various scales are shown by marks

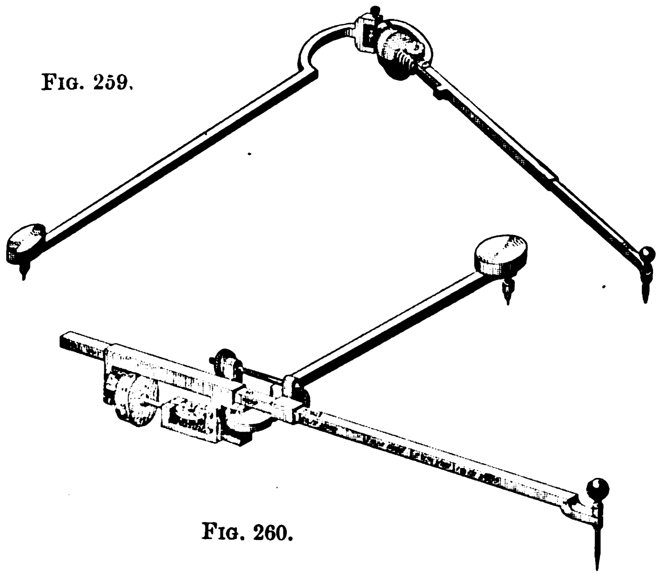

FIG. 259.

FIG. 260.

Amsler Planimeters.

engraved on the bar. There is usually a tangent screw for the accurate adjustment of the sliding frame to these marks.

The form of instrument in which the roller is between the hinge and tracing point is shown in Fig. 259. Here again the roller and hinge are at a fixed distance apart, while the distance from the tracing point to the roller can be altered. The centres of tracing point, roller and hinge lie in one straight line, although the bars are crooked in various ways to comply with the requirements of folding, housing of the roller, &c.

To get accurate results with the planimeter the surface on which

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it works must be smooth and level. In the usual case of a small area the fixed point is set in the paper at some distance outside the boundary of the area and a trial run of the tracing point is made round the boundary to see that every part is reached. The tracing point is then set to the starting mark and slightly pressed into the paper, the instrument is gently raised off the paper at the roller and the roller is turned by finger until the index reads zero. It is then carefully lowered on to the paper again. The tracing point is then guided once round the boundary in a clockwise direction and brought exactly back to the starting point. The index reading then gives the area. As a check on the work, allow the first index reading to remain unaltered and run the tracing point round a second time. The reading now obtained should be double the first reading and equal to twice the area. A large discrepancy would indicate that a mistake had occurred.

Large areas are dealt with by setting the fixed point within the boundary. Then as the tracing point completes the circuit of the boundary the whole instrument swings through a complete revolution about the fixed point. The area is obtained by adding a constant to the index reading. Values of the constant for various scales are engraved on one of the bars.

Areas by Direct Calculation from Field Measurements.—The measurements required for the calculation of the areas of enclosures of various forms are indicated on the figures on pages 314 to 316. For a triangular field the lengths of the three sides may be measured, but the formula for the area in that case does not lend itself to easy calculation. The more usual method for a triangle is to take one of the sides as a base and with the optical square set out a perpendicular from it to the opposite corner. The lengths of the base line and the perpendicular offset from it to the opposite corner are measured and half their product gives the area.

To measure the area of a quadrilateral one of the diagonals may be set out and measured and from it offsets taken to the two opposite corners. The areas of the two separate triangles into which the figure is divided may then be calculated and the whole area obtained. The areas of polygonal figures may also be obtained by dividing them up into triangles and taking sufficient measurements to enable the area of each triangle to be calculated.

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It may sometimes be more convenient in the case of a quadrilateral to measure a long side as a base line, as shown in Figs. 261 and 262, instead of running in a diagonal. Accurately squared offsets are set out and measured to the two corners opposite the base, and the positions at which they occur on the base line are accurately measured. Referring to the figures the area in each case is equal to $\frac{1}{2}$ (y_1 AD + y_2 CB).

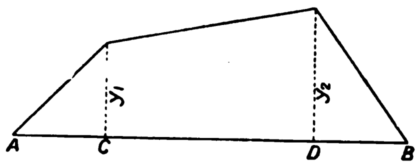

Fig. 261.

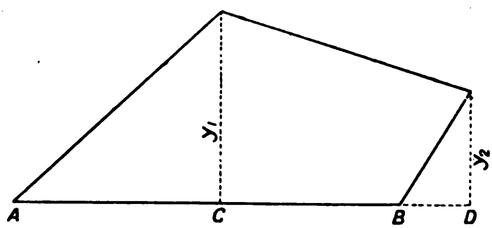

Fig. 262.

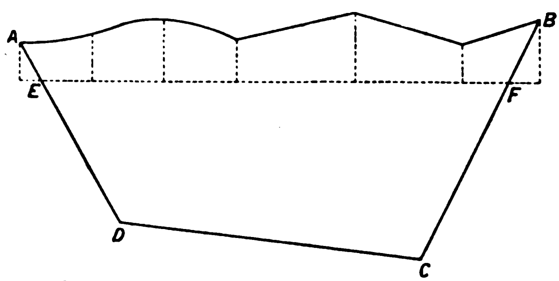

Fig. 263.
Areas from Field Measurements.

A method of dealing with an enclosure having an irregular side is shown in Fig. 263. A base line EF is set out in a convenient position close to the irregular boundary, thus dividing the figure into a quadrilateral EFCD and a strip ABFE. The area of the former may be obtained by either of the methods already described. To obtain the area of the strip offsets are set out from the base line to each change of direction of the boundary and their lengths and positions on the base line are measured. Methods of calculating the area from the offsets have been described in the preceding pages.

Area of a Traverse.—The area contained within the survey lines of a closed traverse may readily be obtained when the co-ordinates of

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the stations have been calculated, and if the survey lines have been used to locate the boundaries of an enclosure by means of offsets then the whole area of the enclosure can be found by calculation from the measurements alone. The method of finding the area of a closed traverse is illustrated in Fig. 264. The stations are numbered in consecutive order one way round and the co-ordinates x_1, y_1, x_2, y_2, &c., are numbered to correspond with the stations. The area of the simple traverse shown will evidently be obtained by measuring the area of the figure A123D and subtracting from it the area of A143D or in detail

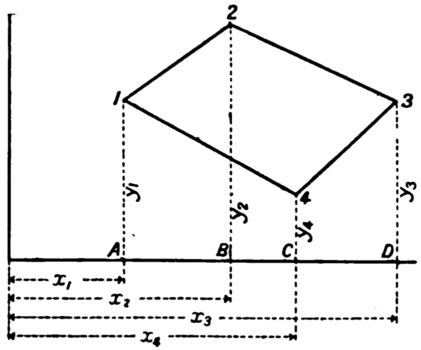

FIG. 264.—Area of a Traverse.

$$\begin{aligned}\text{Area of } 1234 &= \text{A12B} + \text{B23D} - \text{A14C} - \text{C43D} \\ &= \tfrac{1}{2}(x_2 - x_1)(y_1 + y_2) + \tfrac{1}{2}(x_3 - x_2)(y_2 + y_3) \\ &\quad - \tfrac{1}{2}(x_4 - x_1)(y_1 + y_4) - \tfrac{1}{2}(x_3 - x_4)(y_4 + y_3)\end{aligned}$$

This works out to

$$\tfrac{1}{2}[y_1(x_2 - x_4) + y_2(x_3 - x_1) + y_3(x_4 - x_2) + y_4(x_1 - x_3)].$$

The general rule may be expressed in words as follows: Multiply the y-ordinate of each station by x-ordinate-of-the-following-station-minus-x-ordinate-of-preceding-station. Half the sum of these products gives the area.

Correction for Shrinkage of Plan.—Plans drawn on paper are liable to undergo slight alterations in dimensions, which may be contractions or expansions, due principally to atmospheric changes. If there is a scale drawn on the plan it will expand and contract with the paper, and if the dimensions used in calculating areas from the plan are measured by this scale the areas so obtained will require no correction. It is, however, usually more convenient to use a separate boxwood or other scale in taking out areas, and, if the plan

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has altered, the amount of the expansion or shrinkage must be ascertained and an appropriate correction applied to the calculated area.

Let ABCD (Fig. 265) be the size of a square with sides of length s as originally drawn on a plan and let A*bcd* represent its size after shrinkage. Let the amount of the linear shrinkage be the fraction $\frac{1}{x}$ of the original dimensions. The value of $\frac{1}{x}$ will seldom exceed $\frac{1}{400}$. The reduction in the length of each side of the square will be $\frac{s}{x}$, and, as shown in the figure, the reduction of area will consist of a strip $\frac{s}{x}$ in width along each of two sides. The total reduction of area will, therefore, be $2s\,\frac{s}{x}$ or $\frac{2s^2}{x}$. The small square portion at C where the sides overlap should be taken only once, but its amount is quite negligible. The contracted area of the square is, therefore, $s^2 - \frac{2s^2}{x}$ or $s^2\left(1 - \frac{2}{x}\right)$, that is, the area of the square is reduced by the fraction $\frac{2}{x}$ of its original area, so that the fractional contraction or expansion of area on a plan is equal to twice the linear contraction or expansion. The area calculated from a *shrunk* plan must therefore be *increased* by the fraction $\frac{2}{x}$ to arrive at the true area.

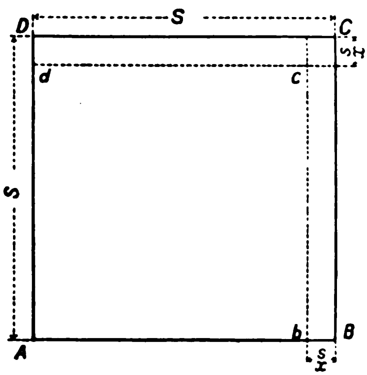

FIG. 265.—Shrinkage of Plan.

Example. The area of a plot of ground is calculated from measurements made with the original boxwood scale used in plotting and works out to 12·386 acres. On comparing the boxwood scale with the scale drawn on the plan it is found that shrinkage has occurred to the extent of 1 ft. in 800 ft. What is the true area of the plot ?

The fractional linear shrinkage is $\frac{1}{800}$, so that the reduction of area is $\frac{1}{400}$, and the calculated area must therefore be increased by $\frac{12 \cdot 386}{400}$ or 0·031 acre. The true area is therefore 12·417 acres.

Areas taken out by planimeter would require to be corrected in the same way.

If expansion or contraction of a plan takes place in one direction only the alteration of area will be fractionally the same as the alteration in linear dimensions.

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CHAPTER XXI

CALCULATION OF EARTHWORK QUANTITIES

THIS chapter deals briefly with the ordinary methods of calculating earthwork quantities which are applicable to classes of work such as cuttings and embankments for roads, railways, canals, and reservoirs, excavations for trenches and foundation pits, and excavations and embankments in levelling off sites for buildings and works.

Purpose of Earthwork Calculations.—Calculations of quantities of earthwork are usually made for one of the following purposes :—

(*a*) In connection with the designing of works, for the purpose of arriving at suitable formation levels for the earthwork, that is, suitable levels to which the surfaces of excavations and embankments are to be finished.

(*b*) For the purpose of obtaining quantities of excavation, embankment, &c., for insertion in a schedule or bill of quantities in connection with work which is to be let out to contract. Such quantities are generally calculated from the information given on plans and sections of the proposed works.

(*c*) In order to obtain final quantities for purposes of payment. Such quantities are measured from the dimensions of the work as executed or authorised.

We have an illustration of the various purposes in the earthwork for a railway. A desideratum in arranging the formation line of the railway is that the resulting quantities of excavation and embankment should balance closely without involving long haulage. The usual procedure is to lay down on the longitudinal section a trial formation line arranged to give equal amounts of excavation and embankment as nearly as can be judged by the eye, and then make a preliminary calculation of the quantities, the result being used to indicate the direction in which adjustment of the formation line should be made in order to obtain closer balance of the earthworks, if required.

When the lines and levels of the railway have been designed, the quantities of excavation and embankment are calculated from the

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plans, sections and cross-sections. If rock is known or believed to occur in the cuttings, estimation is also made of the quantity of rock, and the quantities of soft excavation, rock excavation, and embankment are inserted in the schedule.

As the work proceeds, measurement of the earthwork quantities will be made periodically in order to determine the payments due to the contractor, and the actual total quantities of rock excavation and soft excavation will be determined from measurements made during the progress and after completion of the work.

The unit quantity for the measurement of earthworks in this country is the cubic yard (equal to 27 cub. ft.); where the metrical system of measurement is in use the unit is the cubic metre (1 cub. metre = 35·317 cub. ft.).

Volumes of Solid Bodies.—The following table gives formulæ for finding the volumes of solid bodies of various forms. The fundamental solids, on which the measurement of nearly all earthwork is based, are the prism, wedge, pyramid, and prismoid.

Volumes of Solids.

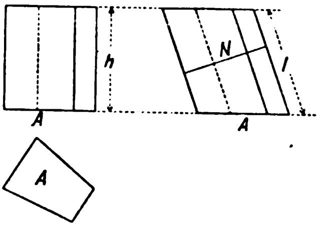
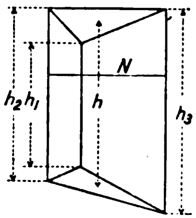
Figure.	Volume = V.
Prism. 	$V = Ah = Nl$ A = Area of Base. N = Area of Section normal to axis.
Truncated Prism. 	$V = Nh$. N = Area of normal section. $h = \frac{h_1 + h_2 + h_3}{3}$. For n-sided prism $h = \frac{h_1 + h_2 + h_3 + \cdots + h_n}{n}$.

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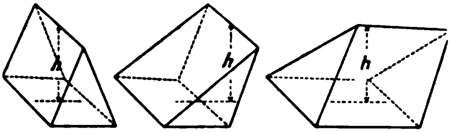
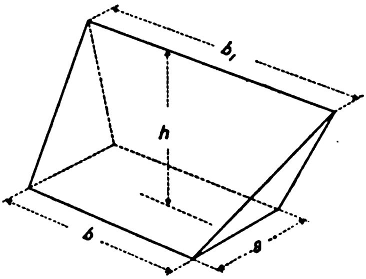
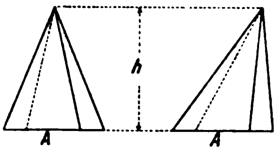
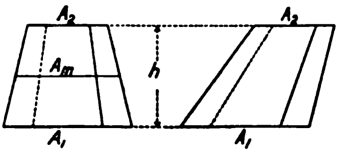
Figure.	Volume = V.
Wedge with parallel sides. 	$V = A\frac{h}{2}$. A = Area of Base. h = Perp. height from base to edge.
Truncated Wedge. 	$V = \frac{ah}{6}(2b + b_1)$.
Pyramid or Cone. 	$V = A\frac{h}{3}$. For cone with base of diam. d— $V = \frac{\pi d^2 h}{12}$.
Truncated Pyramid or Cone. 	$V = \frac{h}{3}(A_1 + A_2 + \sqrt{A_1 A_2})$. $V = \frac{h}{6}(A_1 + 4A_m + A_2)$ Prismoidal Formula. A_m = Area of section at mid-height.

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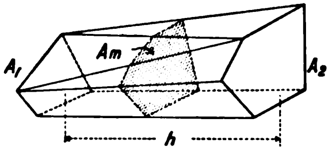
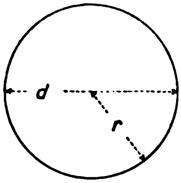
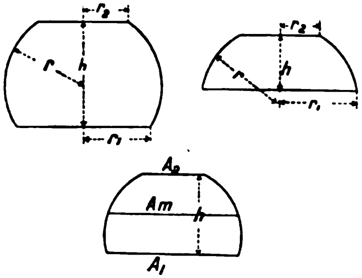
Figure.	Volume = V.
Prismoid. 	$V = \frac{h}{6}(A_1 + 4\,A_m + A_2).$ A_1 and A_2 are the end areas; A_m the area of section midway between the ends.
Sphere. 	$V = \frac{4}{3}\pi r^3 = \frac{1}{6}\pi d^3.$
Frustum of Sphere. 	$V = \frac{\pi h}{6}(3r_1^2 + 3r_2^2 + h^2).$ $V = \frac{h}{6}(A_1 + 4\,A_m + A_2).$

Prismoidal Formula.—The following is one definition* of a prismoid: "A prismoid is a solid having for its two ends any dissimilar parallel plane figures of the same number of sides, and all the sides of the solid plane figures also." According to this definition, the figure shown above under the heading "Prismoid" is not a prismoid. It is shown, however, as the general type of solid which in the calculation of earthwork is usually known as a prismoid, and for which the prismoidal formula gives the exact volume. The prismoidal formula also gives the correct volume for

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any solid having parallel plane ends and with sides formed by moving a straight line around the perimeters of the ends as directrices. It also gives the exact volume for the prism, wedge, pyramid, cone, sphere, spheroid, paraboloid, or for a frustum of any of these.

Where the ground is of fairly uniform surface, the portion of an excavation or embankment, such as for a road or railway, contained between two parallel cross-sections a short distance apart, is for all practical purposes a prismoid, and its volume will be most accurately obtained by the prismoidal formula.

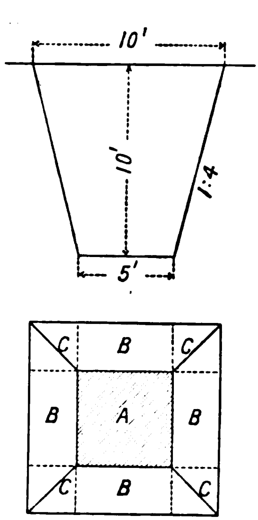

Fig. 266.

Excavation in Foundation Pit.—As an instructive example in the fundamentals of earthwork measurement we may take the case of the excavation for a foundation pit or sewer manway excavated with sloping sides. Bottom area 5 ft. by 5 ft., depth 10 ft., side slopes $\frac{1}{4}$ to 1 (4 vertical to 1 horizontal).

The excavation is in the form of a truncated pyramid, but taken in detail it may be considered, as shown in Fig. 266, as made up of a central prism A, 10 ft. deep by 5 ft. square, four wedges B, with bases 5 ft. by $2\frac{1}{2}$ ft. and height 10 ft., and four corner pyramids C, with bases $2\frac{1}{2}$ ft. square and height 10 ft.

For purposes of comparison we shall work out the volume in detail from the several portions enumerated above, and also by the formula for truncated pyramid, by the prismoidal formula, and by approximate methods which are in common use, known as the "average end area" and "mean area" methods.

In detail :

Central prism - $10 \times 5 \times 5 = 250$ cub. ft.

Four side wedges - $4 \times 5 \times 2\frac{1}{2} \times \frac{10}{2} = 250$,,

Four corner pyramids $4 \times 2\frac{1}{2} \times 2\frac{1}{2} \times \frac{10}{3} = 83\frac{1}{3}$,,

Total $= 583\frac{1}{3}$ cub. ft.

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By formula for truncated pyramid :

$$V = \frac{h}{3}(A_1 + A_2 + \sqrt{A_1A_2}).$$

$$A_1 = 5 \times 5 = 25 \text{ sq. ft.} \quad A_2 = 10 \times 10 = 100 \text{ sq. ft.}$$

$$V = \frac{10}{3}(25 + 100 + \sqrt{25 \times 100}).$$

$$= \frac{10}{3}(125 + 50).$$

$$= \frac{10}{3} \times 175 = 583\tfrac{1}{3} \text{ cub. ft.}$$

By prismoidal formula :

$$V = \frac{h}{6}(A_1 + 4\,A_m + A_2).$$

$$A_m = 7\tfrac{1}{2} \times 7\tfrac{1}{2} = 56\tfrac{1}{4} \text{ sq. ft.}$$

$$V = \frac{10}{6}(25 + 225 + 100).$$

$$= \frac{3{,}500}{6} = 583\tfrac{1}{3} \text{ cub. ft.}$$

The three foregoing methods give the same result, which is the mathematically exact volume of the geometrical solid.

By "Average End Area" Method.—By this method when A_1 and A_2 are the two end areas and h is the distance between them, the approximate quantity is given by the formula $V = \frac{1}{2}(A_1 + A_2)h$.

In the case under consideration

$$A_1 = 5 \times 5 = 25 : \quad A_2 = 10 \times 10 = 100 : \quad h = 10$$

$$V = \tfrac{1}{2}(25 + 100)\,10 = 625 \text{ cub. ft.}$$

The error in this case is $+ 41\tfrac{2}{3}$ cub. ft.

By "Mean Area" Method.—The volume is in this case taken as being equal to that of a prism of height h and area A_m equal to the area of a section midway between the ends, that is

$$V = A_m h.$$

$$A_m = 7\tfrac{1}{2} \times 7\tfrac{1}{2} = 56\tfrac{1}{4} \text{ sq. ft.} : \; h = 10 \text{ ft.}$$

$$V = 56\tfrac{1}{4} \times 10 = 562\tfrac{1}{2} \text{ cub. ft.}$$

The error by this method is —20·83 cub. ft.

A comparison of the results of the foregoing calculations shows that the "average end area" method gives too great a quantity,

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while that obtained by the "mean area" method is too small. The amount of the error by the "average end area" method is twice that by the "mean area" method, the errors in this particular case being about + 7 per cent. and — $3\frac{1}{2}$ per cent. respectively. The large error in this case is due to the great relative difference between the end areas, the one area being four times as large as the other. Where the end areas are more nearly equal the error by the approximate methods becomes much less. Where one end area is 50 per cent. larger than the other the error by the "average end area" method is less than 1 per cent.

Excavation and Embankment for Pond.—As a simple example involving the balancing of earthwork let us take the following: A rectangular pond is to be formed by excavating to a depth of $7\frac{1}{2}$ ft. on level ground, the size at the bottom being 100 ft. by 120 ft.,

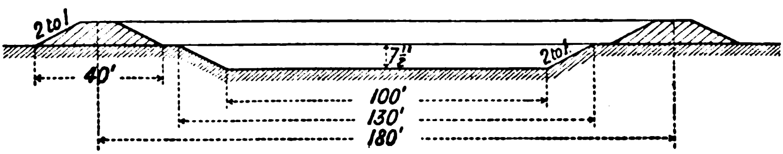

FIG. 267.—Earthwork for Pond.

and the slopes of the sides 2 to 1. What is the quantity of excavation? The excavated material is to be used to form a bank around the pond of a width of 40 ft. at the base and with side slopes of 2 to 1, a bench 5 ft. wide being left all round the excavation, as shown in Fig. 267. To what height can the bank be formed?

The prismoidal formula should be used for finding the quantity of excavation.

$$\begin{aligned}
\text{Bottom area } A_1 &= 100 \times 120 = 12{,}000 \text{ sq. ft.} \\
\text{Top area } A_2 &= 130 \times 150 = 19{,}500 \text{ sq. ft.} \\
\text{Middle area } A_m &= 115 \times 135 = 15{,}525 \text{ sq. ft.}
\end{aligned}$$

$$\text{Volume } V = \frac{7\frac{1}{2}}{6}(12{,}000 + 62{,}100 + 19{,}500)$$

$$= 117{,}000 \text{ cub. ft.} = 4{,}333 \text{ cub. yards.}$$

Width between centre lines of banks, 130 + 10 + 40 = 180 ft.

Length between centre lines of banks, 150 + 10 + 40 = 200 „

Total length of bank measured on centre line = 2 (180 + 200) = 760 ft.

Total volume of bank = 117,000 cub. ft.

$$\therefore \text{ Cross-section area of bank} = \frac{117{,}000}{760} = 154 \text{ sq. ft.}$$

Let x = height of bank.
Then $40 - 2x$ = average width
and area $= x\,(40 - 2x)$

$$\therefore\ x\,(40 - 2x) = 154,$$

and solving the quadratic equation we get $x = 5{\cdot}2$ ft.

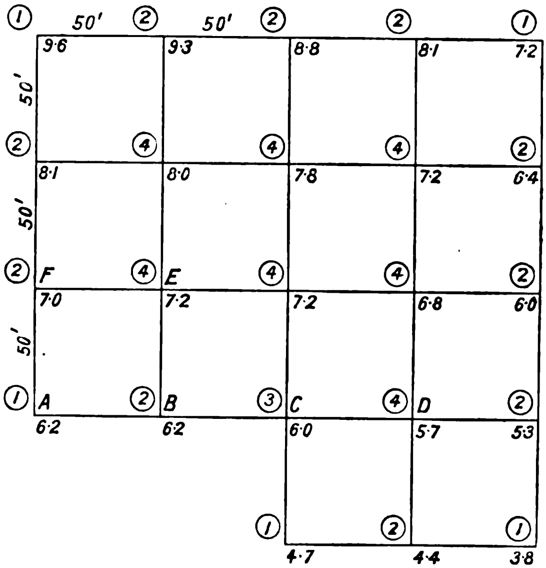

FIG. 268.—Earthwork in Levelling an Area.

Quantity of Earthwork in Levelling an Area.—Fig. 268 illustrates a method of calculating the quantity of excavation or embankment involved in levelling off a building site. The calculation is based on the formula given on p. 331 for a truncated prism. The area to be levelled off is divided up into equal squares or rectangles of a convenient size, and such that the surface of the ground in each of these areas is practically a plane surface. Levels are taken at the corners of the areas and from these, knowing the final level to which the earthwork is to be formed, the depths of excavation or embankment at the corners of the areas are calculated and written down on a plan or diagram. If we take the area ABEF on the diagram with corner heights h_1, h_2, h_3, and h_4 of 7·0, 7·2, 6·2 and 6·2 ft. respectively, the volume will be given by the formula

$$V = \frac{A}{4}(h_1 + h_2 + h_3 + h_4),$$

$$\text{or } V = \frac{50 \times 50}{4}(7{\cdot}0 + 7{\cdot}2 + 6{\cdot}2 + 6{\cdot}2) = 16{,}625 \text{ cub. ft.}$$

We might calculate the volume of each prism separately in this manner and add the results together to obtain the total volume, but this would be a laborious process. A little consideration will show that in summing the volumes of all the prisms, the depth at A (6·2 ft.) will have been taken only once; the depth at B (6·2 ft.) which is common to two prisms will have been taken twice; the depth at C, common to three prisms, will have been taken three times and the depth at points such as D, which is the common corner of four prisms, will have been taken four times, the number of times for each corner of the diagram being as indicated by the figures in small circles. We may, therefore, avoid the separate calculation of the volumes of the prisms if we take the sum of the corner depths multiplied each by its circled number, the total sum so obtained being multiplied by one-fourth of the base area of a prism to give the total volume.

Thus for the case shown on the diagram the total volume will be

$$V = \frac{50 \times 50}{4}(9{\cdot}6 + 2 \times 9{\cdot}3 + 2 \times 8{\cdot}8 + 2 \times 8{\cdot}1 + 7{\cdot}2 + 2 \times 8{\cdot}1 + 4 \times 8{\cdot}0 + 4 \times 7{\cdot}8 + 4 \times 7{\cdot}2 + 2 \times 6{\cdot}4 + 2 \times 7{\cdot}0 +, \text{\&c.}),$$

giving, when extended to include all the corners, a total of 242,687 cub. ft. or 8,988 cub. yards.

It will seldom happen that an area to be excavated or levelled off can be divided into an exact number of equal small areas as in the foregoing example. Where the boundaries are irregular the area may be divided up by two sets of parallel lines at right angles to each other, forming a number of rectangles in the interior of the area and a number of irregular figures around the boundaries. The volume included in all the complete rectangular prisms will be obtained by the method just given, and for the other irregular prisms around the boundary, the volume of each will be separately

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obtained by multiplying its base area by the mean of its corner heights.

Where the ground surface is irregular or where the earthwork is to be finished off with sloping sides or to an irregular formation, the quantity of earthworks should preferably be taken out from cross-sections plotted on paper to a sufficiently large scale.

Earthwork for Roads, Railways, &c.—The general forms of the cuttings and embankments for railways and roads are as shown in Fig. 269. The formation width W and the angle of slope of the sides are usually constant, while the depth D and the inclinations of the ground surface vary. For a cutting or embankment of constant formation width and constant side slope, situated on ground which is level transversely, the area of cross-section can be calculated at any point provided the depth is known. Let the tangent of the angle which the slope makes with the vertical be S (for slope of $1\frac{1}{2}$ to 1, S = $1\frac{1}{2}$; for slope of 2 to 1, S = 2), then

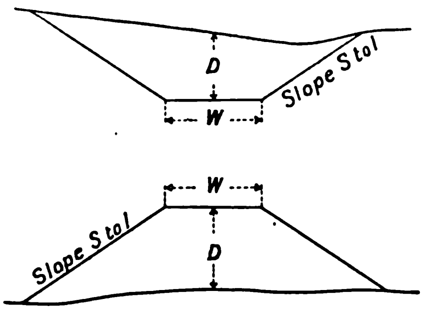

FIG. 269.—Railway Cutting and Embankment.

$$\begin{aligned}\text{Width at formation} &= W\\ \text{Width at ground} &= W + 2SD\\ \text{Average width} &= W + SD\\ \text{Area of section} &= D(W + SD).\end{aligned}$$

Take an embankment for a double-line railway 30 ft. wide at formation, 10 ft. deep, and with slopes of $1\frac{1}{2}$ to 1

$$\text{Area of section, } 10(30 + 1\tfrac{1}{2} \times 10) = 450 \text{ sq. ft.}$$

We get from this a method of finding the quantities in cuttings and embankments from the longitudinal section, which will give fairly accurate results if the ground is nearly level across the line. The method is useful for making preliminary estimates of quantities,

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and is illustrated in detail in the following example, which refers to the embankment shown on the longitudinal section in Fig. 270.

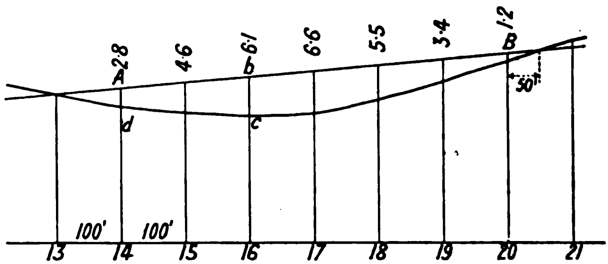

FIG. 270.—Earthwork in Railway Embankment.

The formation width of the double-line railway is 30 ft. and the side slopes are $1\frac{1}{2}$ to 1. The calculations are shown in the following table :—

CALCULATION OF QUANTITY OF EMBANKMENT.

Cross-Section.	D	W + $1\frac{1}{2}$D.	Area D(W + $1\frac{1}{2}$D).	Distance.	Quantity.
	Ft.	Ft.	Sq. ft.	Ft.	Cub. ft.
13	0·0	30	0	50	
14	2·8	34·2	96	100	
15	4·6	36·9	170	100	
16	6·1	39·1	238	100	
17	6·6	39·9	263	100	109,600
18	5·5	38·2	210	100	
19	3·4	35·1	119	100	
20	1·2	31·8	38	75	2,850
20^{50}	0·0	30·0	0	25	
					112,450
			Total Quantity		4,165 cub. yards

In the above table the quantity is calculated on the assumption hat each cross-section applies to a length on either side of it extend-

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ing halfway to the next cross-section. The result arithmetically is exactly the same as if the quantities were taken out by average end areas, but the setting down of the figures is simplified. Where a series of cross-sections have to be multiplied by the same common distance, then instead of finding the separate products and adding them together, add up the cross-section areas and multiply their sum by the common distance as is done in the foregoing table.

Where the surface of the ground is sloping or irregular the quantities will be best obtained from plotted cross-sections whose areas may be found by planimeter or by scaling and computation. In practice the average end area method of arriving at the volume is generally considered quite satisfactory and the prismoidal method is rarely applied, on account of the extra labour involved.

Prismoidal Method.—To illustrate the prismoidal method applied to a continuous excavation or embankment, we shall take the portion AB shown on Fig. 270, which consists of three portions 200 ft. long, each portion having two end areas and a middle area. We shall assume that the areas given in the table (p. 340) are the actual measured areas of the cross-sections.

The prismoidal formula for a portion such as *Abcd* is

$$V = \frac{h}{6}(A_1 + 4A_2 + A_3),$$

where A_1, A_2, and A_3 are the front, mid and end areas respectively. If A_3, A_4, and A_5 are the areas of the next portion its volume will be $= \frac{h}{6}(A_3 + 4A_4 + A_5)$ and so on for successive portions. If these volumes are added together we get the general formula for the total volume

$$V = \frac{h}{6}(A_1 + 4A_2 + 2A_3 + 4A_4 + 2A_5, + \ldots + A_n)$$

where h is the length of each assumed prismoid, that is twice the interval between cross-sections. If l is the common interval between sections $\left(l = \frac{h}{2}\right)$, the formula will be

$$V = \frac{l}{3}(A_1 + 4A_2 + 2A_3 + 4A_4 + 2A_5 + \ldots + A_n).$$

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Applying the formula to the portion AB we have

$$V = \frac{100}{3}(96 + 4 \times 170 + 2 \times 238 + 4 \times 263 + 2 \times 210 + 4 \times 119 + 38) = \frac{100}{3} \times 3{,}238 = 107{,}933 \text{ cub. ft.}$$

$$\text{End portion at A} = \frac{96 \times 100}{2} = 4{,}800 \text{ cub. ft.}$$

$$\text{End portion at B} = 38 \times \frac{50}{2} = 950 \quad ,,$$

$$\text{Total embankment} = 113{,}683 \text{ cub. ft.}$$

$$= 4{,}210 \text{ cub. yards.}$$

CHAPTER XXII

ADJUSTMENT OF INSTRUMENTS

Dumpy Level.—In all forms of Dumpy level there is provision for adjusting the line of sight by raising or lowering the diaphragm containing the cross hairs. Generally, also, there is provision for making adjustment between the bubble tube and the telescope and for making adjustment between the telescope and the stage, as indicated in Fig. 195. In some levels, however, the bubble tube is fixed with respect to the telescope, and in others the telescope is fixed with respect to the stage, or, what amounts to the same thing, the stage is dispensed with and the telescope is directly and solidly connected to the vertical axis.

The purpose of making adjustments is that the following requirements of a correct level may be attained :—

(*a*) The line of sight of the telescope should be parallel to the bubble axis.

(*b*) Both the line of sight of the telescope and the axis of the bubble should be at right angles to the vertical axis of rotation of the level.

In the case of the ordinary Dumpy level, with means of adjustment provided at three points, as above mentioned, the attainment of requirement (*a*) can for most practical purposes be effected by making adjustments at any two of the three points. It is better, however, to avoid altering the cross hairs, assuming that these have been correctly placed by the maker so that the line of sight and line of travel of the centre of the object glass are in coincidence. The adjustment of the ordinary Dumpy level to effect compliance with requirement (*a*) are, therefore, made by manipulation of the screws connecting the bubble tube to the telescope and the screws connecting the telescope to the stage.

Peg Method of Adjustment to make the Line of Sight parallel to the Bubble Axis.—The process known as the peg method is illustrated in Fig. 271. The level is planted midway between two pegs or

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firm, definite points A and B, say, 400 ft. apart, and levelled up. Readings are taken to a staff held at points A and B, the bubble being brought exactly to the centre of its run for each reading. The difference of these readings will give the true difference of elevation of the two points, as although the line of sight may not be horizontal, the error in the readings will be the same in each case since the distances are equal.

Referring to Fig. 271, the lines of sight from the level at position C_1 will cut the staffs at points a and b, which will be at the same level, so that the difference of the readings Aa and Bb will be the correct difference of level of the points A and B. The instrument is now transferred and set up at a point C_2, a short distance beyond one of the two pegs and nearly in line with them. Make the distance

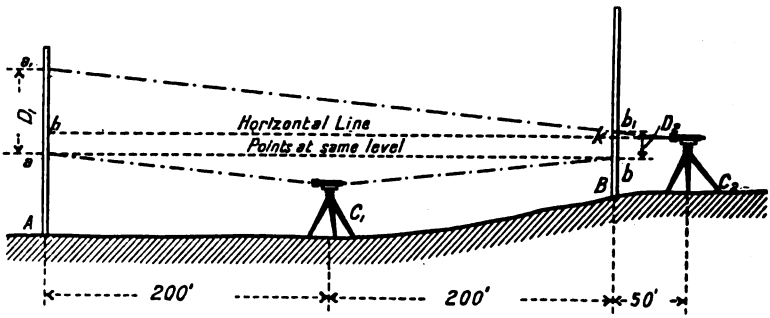

FIG. 271.—Adjustment of Level.

BC_2 an even fraction of the distance AB, say, 50 ft. The instrument having been levelled and the bubble brought to the centre readings are again taken to the staffs at A and B, the readings being represented by Aa_1 and Bb_1 respectively. If the level is in correct adjustment, points a_1 and b_1 will be at the same level, or D_1 and D_2, the differences of the staff readings on A and B respectively, will be the same. If D_1 and D_2 are not the same the instrument is incorrect and the amount of error of the line of sight in the length AB will be equal to $D_1 - D_2$. By proportion we get the amount of error in the distance AC_2 as equal to $(D_1 - D_2) \times \frac{AC}{AB}$. Similarly, the amount of error in the length BC_2 is equal to $(D_1 - D_2) \times \frac{BC_2}{AB}$. The amounts are represented by the distances a_1h and b_1k

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on the figure respectively. If we subtract the error a_1h, calculated as above from the staff reading Aa_1, we get the reading Ah, which a correct level would give.

The levelling screws are, therefore, turned so as to make the cross hair read the calculated value Ah. This will have the effect of throwing the bubble out of centre. The bubble is brought back to the centre of its run by raising or lowering one end by means of the adjusting screws connecting its casing to the telescope, care being taken not to disturb the reading on the staff. A reading should also be taken on the staff at B, and if the value obtained is equal to the former reading Bb_1 increased or diminished by the calculated error b_1k, the bubble remaining central, we may take it that the requirement (*a*) has been fulfilled.

An example worked out will make the method more explicit.

Points A and B were 400 ft. apart, and C_2 was 50 ft. beyond B. The readings and calculations were as follows :—

	Reading on A.	Reading on B.
Level at C_1 .	5·28	3·75
Level at C_2 .	7·51	5·66
	$D_1 = -2{\cdot}23$	$D_2 = -1{\cdot}91$

$$D_1 - D_2 = 0{\cdot}32 = \text{Error in AB.}$$

$$\text{Error in } AC_2 = 0{\cdot}32 \times \frac{450}{400} = 0{\cdot}36$$

$$\text{Correct staff reading at A} = 7{\cdot}51 - 0{\cdot}36 = 7{\cdot}15$$

$$\text{Error in } BC_2 = \frac{0{\cdot}32 \times 50}{400} = 0{\cdot}04$$

$$\text{Correct staff reading at B} = 5{\cdot}66 - 0{\cdot}04 = 5{\cdot}62.$$

The telescope is, therefore, tilted by means of the levelling screws till the reading on staff at A is reduced from 7·51 to 7·15. The line of sight is thus made horizontal and the bubble is then brought to the centre of its run by means of its adjusting screws.

If, in making the subtractions to obtain the differences of the staff readings, D_1 and D_2 come out of opposite sign, the error in length AB will be the *arithmetical sum* of the values D_1 and D_2.

To make the Line of Sight and Bubble Axis perpendicular to the Vertical Axis.—The further adjustment required to make the

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line of sight and the axis of the bubble perpendicular to the vertical axis is effected by means of the screws connecting the telescope to the stage. The instrument is levelled up, the telescope is placed over a pair of opposite screws, and by turning these the bubble is brought exactly to the centre. The telescope is rotated through 180° on the vertical axis so as to point in the opposite direction. If the bubble is still central its axis is at right angles to the vertical axis. If the bubble has deviated from the centre it should be brought halfway back by the levelling screws and the rest of the way by the adjusting screws connecting the telescope to the stage. The process will be repeated and further slight adjustment made, if necessary, till the bubble is found to remain central when the instrument is turned in any direction.

Adjustments when the Bubble is permanently fixed to the Telescope.—In this case it is better to do first the adjustment for making the bubble axis perpendicular to the vertical axis by the method described in the preceding paragraph. The peg method is used for making the line of sight parallel to the bubble axis. Referring to Fig. 271, after the readings on the pegs have been taken from position C_1, the instrument is set up at C_2 and levelled accurately. Readings are taken to the staffs at A and B and if this shows that the line of sight is not horizontal, the correction is made by raising or lowering the diaphragm containing the cross hairs. If, as in the example worked out, a lower reading is required on the staff at A to give a horizontal line of sight then the diaphragm must be raised; to give a higher reading the diaphragm must be lowered. To raise the diaphragm the lower capstan screw must first be loosened and the upper screw then tightened, and *vice versâ* to lower the diaphragm. When the line of sight has been altered so as to give the true difference of elevation of the pegs the adjustment is complete. Care must be taken to see that the bubble is kept central.

Adjustments when the Telescope is firmly fixed to the Vertical Axis.—In this case the bubble axis must first be made perpendicular to the vertical axis. This is done by setting up and levelling the instrument, placing the telescope over a pair of screws, bringing the bubble exactly to the centre, reversing the telescope end for end and correcting the displacement of the bubble one half by the levelling screws and the other half by the adjusting screws connecting the bubble to the telescope, the process being repeated till the bubble remains central for any position of the telescope.

The remainder of the adjustment, to make the line of sight parallel to the bubble axis is accomplished by the peg method and the raising or lowering of the diaphragm exactly as described in the foregoing paragraph.

Wye Level.—In the Wye level the diaphragm containing the cross hairs is adjustable in two directions at right angles to each other, the means of adjustment being provided in order that the line of sight of the telescope may be made to coincide with the axis of the circular collars of the telescope.

The adjusting screws connecting the bubble tube to the telescope enable the axis of the bubble tube to be made parallel to the supporting surfaces of the wyes on which the collars rest. If the collars are of equal diameter, as they should be, the bubble axis will then be also parallel to the axis of the collars.

The means of adjustment provided between the telescope and the stage enables the bubble axis and line of sight to be made perpendicular to the vertical axis of the instrument.

Adjustment of the Cross Hairs.—Also known as the adjustment for collimation. The telescope is sighted and focussed on to a well-defined point, and carefully adjusted by means of the levelling screws till the intersection of the cross hair with the vertical hair is exactly on the point. The stage is clamped, the clips are loosened, and the telescope is rotated halfway round so as to bring the bubble from the top position to the bottom or *vice versâ*. If the intersection of the hairs has deviated from the mark, bring the horizontal cross hair halfway back by means of the vertical pair of diaphragm capstan screws and the vertical cross hair halfway back by means of the horizontal pair of screws. Then sight again on the mark and test by again turning the telescope half round, and adjusting, if necessary, till the centre of the cross hairs appears to remain fixed, as the telescope is rotated through a revolution. The line of sight is then parallel to and practically in coincidence with the axis of the circular collars of the telescope. Where there are two vertical cross hairs and a single horizontal hair giving two intersections, both intersections should describe a circle about the point midway between them as centre.

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To make the Axis of the Bubble Tube parallel to the Line of Supports of the Wyes.—The instrument having been levelled up, the telescope is placed over a pair of screws and clamped in position and the bubble is brought exactly to the centre. The clips are then loosened, the telescope is taken out and reversed end for end in the collars. If the bubble is now displaced from the centre, the axis of the tube is not parallel to the supports. The correction is made by bringing the bubble halfway back by the levelling screws and the rest of the way by the capstan screws connecting the bubble tube to the telescope. Having made the adjustment, repeat the test and correct again if necessary.

The adjustment of the bubble tube and the adjustment of the cross hairs having been made, the line of sight and the axis of the bubble tube will now be parallel, provided the collars are of the same diameter. If there is any doubt as to this the adjustment to make the bubble axis parallel to the line of sight should be made by the peg method, as described for the Dumpy level.

To make the Axis of the Bubble Tube perpendicular to the Vertical Axis of the Instrument.—The instrument having been set up and levelled, the telescope is placed over a pair of levelling screws and the bubble is brought exactly to the centre. Rotate the telescope through 180° about the vertical axis so that it is turned end for end over the same pair of screws. If the bubble has deviated from the centre the axis of the bubble tube is not at right angles to the vertical axis. To make the correction, bring the bubble halfway back by the levelling screws and the remainder of the way by the adjusting screws connecting the wyes to the stage. Repeat the test and again adjust, if necessary, till the bubble is found to remain central for any position of the telescope.

Theodolite.—The following adjustments are required in a theodolite to be used for reading horizontal angles :—

(1) The adjustment of the levels on the vernier plate so that the axes of their bubble tubes shall be perpendicular to the vertical axis of the instrument.

(2) The adjustment of the cross hairs so that the line of sight of the telescope shall be perpendicular to the transit axis and shall thus describe a plane as the telescope is transited.

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(3) The adjustment of the supports on top of the standards so that the transit axis shall be truly horizontal and the line of sight shall revolve in a vertical plane when adjustments Nos. (1) and (2) have been effected and the instrument has been levelled up.

The following additional adjustments are required if the instrument is to be used for levelling and for reading vertical angles :—

(4) The adjustment of the level on the telescope so that the axis of its bubble tube shall be parallel to the line of sight.

(5) The adjustment of the vernier arms of the vertical circle so that the reading shall be zero when the line of sight is horizontal.

Adjustment of the Plate Levels so that the Axes of their Bubble Tubes shall be perpendicular to the Vertical Axis of the Instrument.— Set up the instrument on firm ground with the lower clamp loose and the upper clamp fixed. Set the longer of the two levels parallel to a pair of diagonally opposite screws, or in the case of a three-screw instrument over any pair of screws. Turn the levelling screws till both bubbles are brought to their centres. Then turn the head of the instrument through 180° so that each level is reversed end for end and is over the same screws as before. If the bubbles remain central the levels are in correct adjustment. If not, adjust first the longer level by raising or lowering one end by means of the capstan nuts (*c*, *c*, Fig. 127) which connect it to the vernier plate, so as to bring the bubble halfway back to the centre, the bubble being brought back the rest of the way by turning the levelling screws. Repeat the test for this level by turning the head back through 180° and noting if the bubble now remains central, and adjust again if necessary. When the longer level has been brought to satisfactory adjustment in this manner the shorter one may be simply adjusted by raising or lowering one end by means of its capstan nuts till the bubble is central simultaneously with the bubble of the longer level.

Adjustment of the Cross Hairs.—The diaphragm of the theodolite telescope is usually fixed by two pairs of capstan screws, one in the vertical direction and the other in the horizontal direction. Sometimes, as in the instrument shown in Fig. 127, the cross hairs are adjustable only in the horizontal direction. For the reading of horizontal angles, prolonging of straight lines and setting out of

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works the accurate adjustment of the cross hairs in the horizontal direction is of prime importance.

To make the horizontal adjustment set up and level the instrument and sight the intersection of the cross hairs exactly on a distant and definite back mark. Transit the telescope and line out an arrow at a considerable distance, say, 300 ft. from the telescope. Loosen the lower clamp, turn the head of the instrument through 180°, sight on the back mark and again transit the telescope. If the line of sight strikes the arrow the cross hairs are in correct horizontal adjustment. If the line of sight deviates from the arrow line out a second arrow opposite the first and measure the distance between them. The correction of the hairs is made by putting in an intermediate third arrow at *one-fourth* of the deviation measured back from arrow No. 2 and bringing the line of sight to strike this arrow by adjusting the diaphragm horizontally. If the line of sight requires to move towards the left in shifting from arrow No. 2 to arrow No. 3 the diaphragm will require to be moved to the right and *vice versâ*. The motion of the diaphragm is effected by first loosening one of the screws (*b*, *b*, Fig. 127) and then tightening the other. Having made the adjustment sight again on the back mark and transit the telescope. The line of sight should now strike *midway* between arrows Nos. 1 and 2. If it does so the horizontal adjustment is correct. If not, repeat the test and make further slight correction till the same line of sight is given on transiting with the telescope normal as with the telescope inverted.

Another method of making this adjustment is as follows: The instrument having been accurately levelled sight on a fine mark, such as the point of an arrow, at a considerable distance, say, 300 ft., making the centre of the cross hairs cut the mark exactly by turning the lower tangent screw. Loosen the clips which hold the ends of the transit axis in their bearings, loosen the opposing screws which fix the vertical clipping arm to the standard, carefully lift the telescope and replace it with the ends of the transit axis reversed. Transit the telescope so as to point again to the sighting mark. If the centre of the cross hairs hits the mark the adjustment is correct. If there is displacement, correct *half* the error in this case by the pair of horizontal capstan screws (*b*, *b*, Fig. 127), and then repeat the test and make further correction, if necessary, till the adjustment is satisfactory. If it is necessary to perform this

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adjustment in a limited space a fine black mark on a piece of white paper fixed at a distance of 30 or 40 ft. will serve quite well as a sighting mark.

Adjustment of the Supports at Top of the Standards.—Set up and level the instrument in a position where some high definite point can be sighted, giving a considerable angle of elevation. With telescope normal sight to the point, depress the telescope and line in an arrow on the ground or make a mark somewhere on the line of sight nearly under the high point. Sight again to the high point with telescope inverted, depress the telescope and note if the line of sight strikes the arrow or mark. If it does so the supports are in correct adjustment and the line of sight revolves in a vertical plane. If the line of sight with telescope inverted deviates from the arrow or mark, put a second arrow or mark into line opposite the first one and plant a third arrow midway between them. A plane through the high point, the axis of the instrument and this third arrow will be a vertical plane, and the adjustment consists in raising or lowering the adjustable support so as to make the line of sight revolve in this vertical plane. The telescope is sighted to the third arrow and tilted up to view the high point. The adjustable support is raised or lowered by means of the opposing screws (*a*, *a*, Fig. 127) till the intersection of the hairs cuts the high point or is exactly on the vertical line through the point. The adjustment should then be correct, as shown by the line of sight striking the middle arrow when the telescope is again depressed. Repeat the test and make further slight correction if required.

Adjustment of the Axis of the Telescope Level parallel to the Line of Sight.—This adjustment is effected by the peg method in the manner described for the Dumpy level.

Adjustment of Vertical Circle Verniers.—In order that an angle of elevation or depression from a horizontal plane may be read directly it is necessary that the verniers should indicate zero when the line of sight is horizontal. Set up and level the instrument with the telescope over a pair of screws. Set the vernier index of the vertical circle to zero. If the telescope bubble is not central, turn the opposing screws attaching the vertical clipping arm of the vernier to the standard till the bubble is brought exactly to the

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centre of its run. The motion of the opposing screws on the clipping arm causes the vernier arms, vertical circle and telescope to rotate in the vertical plane so that the vernier reading remains at zero, and when the bubble is brought to the centre the line of sight is horizontal and an angle of elevation or depression may be read from that position. The adjustment is complete when the telescope bubble remains central, as the head of the instrument is rotated. To effect this, having got the bubble central for one position, as above described, rotate the head through 180°. The bubble may now have departed from the centre. If so, correct half the error by the levelling screws and the other half by the opposing screws attaching the clipping arm to the standard. Perform the operation again, starting with the telescope at right angles to its first position and repeat, if necessary, till the bubble remains central as the head is rotated.

In some instruments, such as Cooke's pattern of transit theodolite, illustrated in Fig. 127, the long sensitive level, instead of being placed on the telescope is attached to the vernier arm. In this case the level must be adjusted so that "when the central line of vision of the telescope is horizontal, and the zero lines of the vertical verniers coincide with the zero diameter of the vertical circle, the bubble may be in the middle of its run." The following is the method of adjustment recommended by Messrs. Cooke :—

Having levelled the instrument carefully by means of the bubbles on the horizontal plate, bring the bubble in the azimuth level to the middle of its run by means of the antagonistic screws *e, e,* at the end of the clipping arm. Now set the zero diameter of the vertical circle to coincide exactly with the zero lines on the vertical verniers and clamp it there. Observe an ordinary levelling staff held at as great a distance as it can be distinctly seen, and take the reading by the horizontal web. Now release the clamp and transit the telescope and again adjust the zero diameter of the vertical circle to the zero lines on the verniers. Revolve the head in azimuth one-half turn, bringing the telescope to its former position, and once more take the reading of the staff. If it is not the same as previously observed correct half the error by the antagonistic screws at the end of the clipping arm and then repeat the operation until all error is by this means eliminated. When the adjustment is complete correct the azimuth level by means of the capstan headed locknuts *d, d,* so that the bubble remains in the middle of its run.

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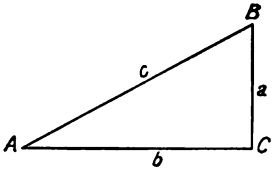
APPENDIX.

Geometric and Trigonometric Formulæ.

Right Angle Triangle.



1. $c^2 = a^2 + b^2$.
2. $c = \sqrt{a^2 + b^2}$.
3. $b^2 = c^2 - a^2$.
4. $b = \sqrt{(c + a)(c - a)}$.
5. $a^2 = c^2 - b^2$.
6. $a = \sqrt{(c + b)(c - b)}$.
7. $\text{A} + \text{B} = \text{C} = 90°$.
8. $\text{A} = 90° - \text{B}$.

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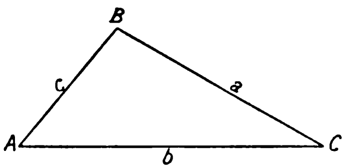
9. $\text{Sin A} = \dfrac{a}{c} = \cos \text{B}$.
10. $\text{Cos A} = \dfrac{b}{c} = \sin \text{B}$.
11. $\text{Tan A} = \dfrac{a}{b} = \text{cot. B}$.
12. $\text{Cosec A} = \dfrac{c}{a} = \sec \text{B}$.
13. $\text{Sec A} = \dfrac{c}{b} = \text{cosec B}$.
14. $\text{Cot A} = \dfrac{b}{a} = \tan \text{B}$.
15. $a = c \sin \text{A} = c \cos \text{B}$.
16. $a = b \tan \text{A} = b \cot \text{B}$.
17. $b = c \sin \text{B} = c \cos \text{A}$.
18. $b = a \tan \text{B} = a \cot \text{A}$.
19. $c = \dfrac{b}{\cos \text{A}} = \dfrac{b}{\sin \text{B}}$.
20. $c = \dfrac{a}{\sin \text{A}} = \dfrac{a}{\cos \text{B}}$.

Oblique Triangles.

21. $\text{A} + \text{B} + \text{C} = 180°$.
22. $\dfrac{a}{\sin \text{A}} = \dfrac{b}{\sin \text{B}} = \dfrac{c}{\sin \text{C}}$.
23. $a = b \cos \text{C} + c \cos \text{B}$.
    $b = C \cos \text{A} + a \cos \text{C}$.
    $c = a \cos \text{B} + b \cos \text{A}$.



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24. $a^2 = b^2 + c^2 - 2bc \cos A.$

25. $\tan A = \dfrac{a \sin C}{b - a \cos C}.$

26. $\sin \dfrac{A}{2} = \sqrt{\dfrac{(s-b)(s-c)}{bc}}$, where $s = \frac{1}{2}(a + b + c)$.

27. $\cos \dfrac{A}{2} = \sqrt{\dfrac{s(s-a)}{bc}}.$

28. $\tan \dfrac{A}{2} = \sqrt{\dfrac{(s-b)(s-c)}{s(s-a)}}.$

Solution of Oblique Triangles.

Given.	Required.	Formulæ.
A, B	C	$C = 180° - (A + B).$
a, b, c	A	Nos. 26, 27, or 28. The calculation is most easily made by No. 27, $\cos \dfrac{A}{2} = \sqrt{\dfrac{s(s-a)}{bc}}.$ $\text{Sin } A = \dfrac{2}{bc}\sqrt{s(s-a)(s-b)(s-c)}.$
C, a, b	c	No. 24, $c = \sqrt{a^2 + b^2 - 2ab \cos C}.$
	A	$\text{Sin } A = \dfrac{a \sin C}{c}.$
	B	$\text{Sin } B = \dfrac{b \sin C}{c}$, or $B = 180° - (A + C).$
	Area	$\Delta = \frac{1}{2} ab \sin C.$
A, a, b	B	$\text{Sin } B = \dfrac{b \sin A}{a}.$
	C	$\text{Sin } C = \dfrac{c \sin A}{a}$, or $C = 180° - (A + B).$
	c	$c = \dfrac{a \sin C}{\sin A}.$ Note that B may have two values, one less than 90° and the other greater than 90°, hence C and c may each also have two values.

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Given.	Required.	Formulæ.
B, C, a	A	$A = 180° - (B + C).$
	b	$b = \dfrac{a \sin B}{\sin A}.$
	c	$c = \dfrac{a \sin C}{\sin A}.$
	Area	$\Delta = \dfrac{a^2 \sin B \sin C}{2 \sin A}.$

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