TOPOGRAPHICAL

SURVEYING AND SKETCHING

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INTRODUCTION.

This text book was, for the most part, originally prepared in the form of lectures and pamphlets for the instruction of student officers of the U. S. Army Service Schools at Fort Leavenworth, Kansas, and it has had the test of several years of use, criticism and correction by a most intelligent body of instructors and students.

In "Surveying" no departure from the regular and recognized methods has been attempted. Indeed, at this time, it would be difficult to add anything new or original to the many excellent text books on this subject. Attention is invited, however, to the treatment herein presented of the adjustments of instruments, particularly with reference to the level tube; to the marking of the stadia rod which automatically applies the correction for the constant of the instrument; to the methods of applying latitudes and departures in reducing and plotting a survey and in computing areas; and to the methods of resection with the plane table.

In the subject of "Sketching," by which is meant the application of rapid and approximate methods of surveying with hand instruments, the methods adopted and developed in the Department of Engineering of the U. S. Army Service Schools at Fort Leavenworth are described and explained. By these methods an individual road sketch covers twenty miles of road in a day, and shows topography by 20-foot contours and all essential details to a distance of 400 or 500 yards on each side of the trail. By combining the sketches made on a number of parallel and cross roads a fair topographical map covering an area at least 20 miles square (400 square miles) may be made in one day by a group of sketchers. A day's position sketch covers from one to two square miles with 10-foot contours, and shows all minor incidents and details of the ground. By combining the work of a number of sketchers, the extent of the strip of country that may be covered in a day is limited only by the number of sketchers available, and the resulting map will compare favorably, for all practical purposes, with one made by transit or plane table, and requiring weeks for its completion.

These rather surprising claims would be made with hesitation were it not for the fact that the above stated results have been attained repeatedly in the work at the Service Schools.

The many complete books of tables of logarithms and circular functions, such as Wentworth's, Ludlow's, etc., now available at small cost, make it unnecessary to attach such tables to a text book, and therefore, only a table of reductions of inclined stadia readings to horizontal distance and difference of elevation is appended hereto.

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CHAPTER I.

DEFINITIONS AND PRINCIPLES.

Topography is the determination by observation, measurement, and record on the ground; and the representation by projection and convention on a plane surface, of the forms, features, structures, accidents of surface, and incidents of rock, earth, water and vegetation, of a considered portion of the earth's surface.

Military Topography is the determination and representation of such forms, features, structures, accidents of surface and incidents of land and water as have especial significance in connection with military operations.

The term "topography" is also applied in a general sense to a description of the physical features, collectively, characterizing any region, but in these pages the word will be used in the sense first stated.

The eye of an observer can take in only a limited portion of the region surrounding him. Much is hidden by intervening objects, and of that which he sees he can form only a vague and inaccurate estimate as to relative positions, distances and elevations of the points, objects and features observed. When he leaves the ground he has only his memory of the general appearance of the ground to rely on, and any report which he might make of his observations would necessarily reflect the vagueness and inaccuracies of his impressions, magnified by the usual limitations of memory.

If, however, he has been able on the ground to measure the distances between the most important objects and features, and the directions of the lines joining them, and their relative heights, and has made careful note and record of these measurements, he will take with him information concerning the region covered that is definite and accurate as far as it goes, and that will serve as the basis of a report or description founded on ascertained and recorded facts and not on impressions and memories.

But the same information may be expressed more clearly, accurately and comprehensively, in the form of a drawing than in that of a verbal description, no matter how minute and exhaustive the latter may be. The observer would, therefore, make on paper, to scale, a horizontal projection of the points located by his measurements and would represent incidents of surface, structures and principal features, by the adopted conventions drawn in their proper relative positions with reference to the located and projected or plotted points.

It is evident that in a horizontal projection on a plane surface, elevations cannot be represented graphically. They can be shown, however, by writing at a point, or on a horizontal line, or within a horizontal surface, that is represented on the drawing, the number which expresses in feet the height of the point, line, or surface, respectively, above an assumed horizontal surface called the datum surface.

Such a drawing is called a map, and a complete topographical map will show at a glance the relative positions of all points within its limits, the distance and direction of any two points, one from the other, the actual height above datum of any point, and the relative heights of any two or more points, the slope of the ground at any point, the form and shape of the surface of the ground, the character of the surface and of the vegetation covering it, all occurring incidents of water, and all structures erected by man. Like information could not be conveyed in many pages of verbal description.

Neither could it be acquired by the most careful study of the ground itself. It is a mistake to assume that maps are useful only when the ground itself cannot be seen. No view of the ground from its highest point, no examination of the ground made by riding over all its parts, can give the definite, precise and comprehensive knowledge that may be acquired by the study of a good map. The most complete knowledge is obtained by examination of the ground in connection with study of the map. The first gives general ideas of its appearance, its prominent features, its topographical character, and the nature of soil and vegetation. The second gives precise information of relative positions, distances, directions, elevations, slopes, drainage and topographical features. When both ground and map cannot be studied the map is to be preferred.

The man who has made a survey of a piece of ground, including both field work and plotting, will have more accurate and complete knowledge of that ground than others, however carefully they may have studied both map and ground.

Practice in surveying and sketching is also the best school for training the eye and mind to estimate distances, slopes, elevations, relative positions, topographical forms, etc., and to appreciate the military functions and limitations of the terrain.

In all military operations knowledge of the ground is essential; therefore all military men should have a thorough knowledge of maps and of map making, that is, of topography.

A *map* is a horizontal projection to scale on a plane surface of a considered portion of the earth's surface.

The art of making the observations, measurements, records, and computations that determine directions, distances, elevations, sizes, areas, volumes, and movements on the earth's surface, and of expressing the results in the form of a map or drawing is called *surveying*.

KINDS OF SURVEYS.

Surveys are made by several different means and methods, and for many different purposes. The resulting kinds of survey receive names characteristic of the means or methods employed or the purpose in view.

With reference to the means employed, the survey may be a compass and chain, a plane table, a transit, a stadia, a level, a sextant, or a barometric survey, each made with the instrument named.

With reference to the purpose or object of the survey, there are Land, City, County, Mining, Geodetic, Geographic, Boundary, Hydrographic, Topographic, Geological, Engineering, and Coast Surveys, with others of rarer application.

These pages will concern themselves principally with Topographic Surveying, which is the basis of all others.

Whatever the kind of survey, whether with reference to means employed or object in view, the measurements to be made are those necessary to determine the position of an unknown point with reference to a point whose position is known or is assumed to be known.

SYSTEMS OF COÖRDINATES.

In order to determine the position of a point with reference to a known point, a system of coördinates is necessary. There are three systems of coördinates in general use, all of which are applicable to and used in surveying.

1st. A system of *rectangular coördinates*, in which distances are measured in three directions that are at right angles to each other. Thus, (Fig. 1) from a known point measure north 220 feet, west 300 feet, vertically + 50 feet, and the point at the extremity of the vertical is fully determined with reference to the known point.

2d. A system of *polar coördinates*, in which are measured a horizontal angle, a vertical angle and a distance (radius vector) in the direction determined by the angles. Thus, (Fig. 2) at a known point measure from the north toward the right a horizontal angle of 35°; from the horizontal direction thus determined measure a vertical angle +6°, and in the direction so fixed measure a distance of 300 feet to the required point, which thus becomes fully determined with reference to the known point.

3d. A system of *triangular coördinates*, in which the position of a point is determined with reference to *two* known points. The lines joining the three points (two known and one required) will form a triangle in which three parts may be measured and the other three determined by solving the triangle, thus fixing the position of the unknown point with reference to the two that are known. This system is applied as follows (Fig. 3):

From a known point A as an origin, determine the direction, the horizontal distance and the difference of elevation to a second point B, which thus becomes known The horizontal line between these two known points is called the *base line*, and it is the base of a triangle of which a third point C, whose position is required, is the vertex. If the two angles at the base or the two sides be measured, the triangle may be solved and the location of the vertex or unknown point be determined in its horizontal relations to the two known points.

The elevation of the unknown point with reference to either of the known points may be determined in like manner. The solution of the horizontal triangle has given the horizontal distance of the required point C from the known point A. This is the base of a right-angled triangle, in which the hypothenuse is the inclined line A C joining the points, and the side is the difference of elevation. If the angle at the base be measured, the triangle may be solved and the difference of elevation be determined.

Let A, B and C be points on the surface of the earth. Assume A to be the origin or first known point, B to be the second known point, determined by measuring its direction from A, its horizontal distance A B′, and the difference of elevation B′B. C is a point to be determined by triangular coördinates from A and B. The base line is the horizontal distance (A B′ or A′B) between A and B. Measure the horizontal angles at A and B, and knowing the base and adjacent angles solve the triangle and thus determine the position of C′ or C″ which are in the vertical line through C. Now, knowing A C′, which is the base of the right triangle A C′ C, measure the vertical angle at A, (C A C′) and solve the triangle to determine the side C′ C or difference of elevation. Or knowing B C″, measure the vertical angle at B, (C B C″) and solve for C″ C, the difference of elevation of B and C. In either case the point C becomes fully determined with reference to A or B.

Any two of these three systems may be combined in the same determination by using one system in the horizontal plane, and another in the vertical plane. In the horizontal plane triangular or polar coördinates are generally used, although the rectangular coördinates find frequent application in running short offsets from the main line for the purpose of filling in small details. In the vertical plane, rectangular or triangular coördinates are generally used to determine the difference of elevation. If polar coördinates are used in the vertical plane they are reduced to rectangular coördinates before plotting the work.

A fourth system, which may be called the *three point system* of coördinates, is sometimes used in the horizontal plane. In this system (Fig. 4) three points, A, B and C, must be known. At the unknown or required point D, measure the two angles subtended by the lines A B and B C respectively; that is, the angles A D B and B D C. The angle A D B determines that the point D is on the circumference of a circle of which A B is a chord subtending an angle at the center equal to twice the angle A D B; and the angle B D C determines that the point D is also on the circumference of a circle of which B C is a chord subtending an angle at the center equal to twice the angle B D C. These two intersecting circles being drawn, one point of intersection is found at B and the other will determine the position of D, the required point.

Graphical Method.—Having plotted the known points, A B and C, on paper, use a separate sheet of tracing paper and on it draw through any point three lines making with each other the observed angles A D B and B D C. Place the tracing paper on the plot and move it till its three lines pass through the corresponding points, A, B and C, of the plot. The apex of the angles will then indicate the position of the point D, which may be pricked through and thus plotted.

If the observed angles be set off on a three-arm protractor and the edges of the arms be made to pass through the three plotted points, the center of the protractor will be at the point D.

Analytical solution:

Let D = angle A D B.
D′ = angle B D C.
B = the sum of the two angles at B.
A and C = the angles at A and C and let side A B = d, side B C = d′.

In the quadrilateral the angles at B and D are known, hence

$$A + C = 360^\circ - (B + D + D') = P \qquad (1)$$

$$C = P - A \qquad (2)$$

In the two triangles the side B D is common and

$$BD = \frac{d \sin A}{\sin D} = \frac{d' \sin C}{\sin D'} \qquad (3)$$

From (2). $\sin C = \sin P \cos A - \cos P \sin A$

Substituting in (3).

$$\frac{d \sin A}{\sin D} = \frac{d' \sin P \cos A}{\sin D'} - \frac{d' \cos P \sin A}{\sin D'}$$

$$\frac{d' \sin P}{\sin D'} \cos A = \left\{ \frac{d}{\sin D} + \frac{d' \cos P}{\sin D'} \right\} \sin A$$

$$\text{whence, } \cot A = \cot P \left\{ \frac{d \sin D'}{d' \sin D \cos P} + 1 \right\} \qquad (4)$$

Having found A and C, either of the two triangles may be completely solved and the position of D determined.

This system of horizontal coördinates finds its most frequent application in hydrographic surveying, for locating soundings by reading two sextant angles in the boat. In this case the surface of the water is the datum plane and the vertical coördinate is given by the lead line. It is sometimes useful in topographic surveying, especially with the plane table, as will be explained later.

It will be seen that whatever may be the system of coördinates used, at least three measurements must be made in order to fully determine a point and that these measurements involve direction and distance.

In many surveys, such as land, county, boundary surveys, etc., elevations are disregarded, and only the horizontal coördinates considered. These give sufficient data to project or plot the required points, but the elevations remain unknown and the survey is not a topographical survey.

MAP MAKING.

From a known point as an origin, any number of unknown points within convenient and reasonable distance may be determined. Any one of the determined points may be taken as a new origin of coördinates from which other points may be determined, and so on, until the necessary determinations extend over the area considered.

In topographical surveying, the ultimate object, namely, the production of a map drawn to scale and showing in plan or horizontal projection all of the essential lines and forms that appear on the ground, must be kept constantly in view. All the lines that actually appear on the ground, such as the lines defining roads, trails, railroads, terraces, embankments, cuts, gullies, bluffs, streams, coasts, fences, buildings, etc., may be readily determined by measurements of directions and distances, and as readily plotted on paper by laying off these directions and horizontal distances to suitable scale and drawing the lines so determined.

The determination and representation of the irregular conformation of the surface of the ground in the shape of hills, ridges, knolls, spurs, valleys, ravines, rolling ground, plains, etc., are apparently not so easy, since there are no definite points or lines on the ground by means of which the shape or conformation of the surface may be represented. It is necessary, therefore, to assume a system of imaginary lines on the surface of the ground which will, when reproduced on the map be a geometrical representation of the surface.

Two different systems of lines have been used for this purpose, viz:

1st. The lines of steepest slope. On the map these lines are called hachures.

2d. The lines cut from the surface by horizontal planes at equal vertical intervals. These lines are called contours.

The first system is now little used except sometimes for the purpose of indicating roughly the existence of hill forms in any locality without attempting to represent their actual shape. Formerly this method was much used and careful rules were followed by which different degrees of slope were indicated by different widths of hachure lines. This produced the effect of hill shading and caused the hill masses to appear to stand out in relief on the map. (Fig. 4a.)

By the use of contours the shape of the surface of the ground may be geometrically represented with any desired degree of accuracy, the method being that of "One Plane Descriptive Geometry," in which points and lines are projected on a horizontal plane and elevations are shown by writing at the projected points and on the projected horizontal lines, numbers expressing in feet (or yards or meters) their heights above an assumed horizontal plane called the datum plane.

Numbers thus used are called references, and are usually written in parenthesis to distinguish them from other numbers which may be used on the drawing to express horizontal distances.

Any point that has been determined by measurements on the ground is represented on the map by making a dot at its plotted position, as at a (Fig. 5), and writing near the dot its elevation in parenthesis (326.4). Two other points, similarly determined are shown plotted at b (331.7) and at c (324.1).

A horizontal line is fully represented by drawing its horizontal projection, as a d, and writing on the line its elevation (326.4).

An inclined straight line is fully represented by drawing its horizontal projection and marking the elevations of two of its points. Thus the line b c through the points b (331.7) and c (324.1) represents a fixed inclined line.

To find the elevation of any other point on this line, as the point d, proceed as follows: The rise along the line from c (324.1) to b (331.7) is 7 6 feet and the horizontal distance from c to b scales 196 feet. Therefore at d, which scales 62 feet from c, the rise is $\frac{62}{196} \times 7.6 = 2.4$ feet, and the elevation of d is $324.1 + 2\ 4 = (326.5)$.

To find a point on the line bc having any assumed elevation, say (326.5), proceed as follows: The required point is 2.4 feet above c. The rise from c to b is 7.6 feet in the horizontal distance 196 feet. The point 2.4 feet above c is therefore $\frac{2.4}{7.6} \times 196 = 62$ feet from c. Scale off 62 feet from c and mark the point d thus determined. It is the point on the line bc whose reference is (326.5).

All similar problems may be solved in the same manner by simple proportion, but the graphical solution is much easier and quicker, especially when a number of points on the same line are to be determined.

In Fig. 5 the ruled rectangle represents a piece of paper ruled with two sets of equally spaced lines drawn parallel to the sides of the rectangle. A piece of cross-section paper or of profile paper, machine-ruled by the makers, is suitable. One set of divisions is taken as a vertical scale and is so numbered at the sides of the rectangle as shown, using only the range of numbers that includes the references of the plotted points. Lay the edge of the card through the plotted points b and c; from b follow by eye, using the ruled lines as a guide, a perpendicular to the edge up to the point that reads on the scale (331.7), the reference of b, and dot the point as at b′. From c follow in like manner a perpendicular to the edge up to the point c′ where the reading of the vertical scale is (324.1) the reference of c. Lay a straight edge (ruler or folded edge of paper) through the points b′ and c′.

To find the reference of any point, as d on the line b c, follow the perpendicular through d up to the straight edge at d′ and read the vertical scale. It is found to be (326.4), which is the reference of the point d.

To find a point on the line b c whose reference is (326.4), find on the straight edge the point d′ whose reading on the vertical scale is (326.4) and drop the perpendicular d′d to the edge of the ruled paper. The point d is the required point.

From the points of intersection of the straight edge with the ruled horizontals, drop perpendiculars to the edge of the paper and dot the corresponding points on the line b c. They will be points on b c whose references are (321), (322), (323), etc., to (334) as shown.

When once in position the ruled paper and the straight edge must not be moved until all required points have been properly projected and marked. The projecting lines need not be drawn on the ruled paper. The ruled lines and the spaces between them can be followed by eye with sufficient accuracy, and the same piece of ruled paper can be used for any number of pairs of points.

The dots on the line a (326.4) b (331.7) are projected in the same manner as those on b c and are points on the line a b having references (321), (322), (323), etc. The edge of the ruled paper is, of course, placed on the line a b in determining these points. This is a quick method of "interpolating contour points."

Three points, or a line and a point, or two intersecting lines, or two parallel lines, determine a plane. If the ground surface containing the three points plotted at a, b and c, were a plane surface, that plane is fully determined and represented by the three points a, b and c, or by the line b c and the point a, or by the two intersecting lines b c and b a, or by the two parallel lines b c and a a′.

A plane surface is best represented by drawing its horizontals at equal vertical intervals and marking them with their references. To thus represent the plane determined by the three points a, b and c, and show horizontals at one foot vertical interval, find as already explained points on b c whose references are (321), (322), (323), etc., and on b a points having the same references. Draw straight lines through the pairs of points that have the same references and mark them (321), (322), (323), etc., as shown. These lines will be horizontals or *contours* of the plane. *For any plane* the contours will be straight parallel lines equally spaced.

The ground services are not generally plane surfaces, but rather rounded or curved surfaces with varying slopes and curvatures. If numerous points be determined on the ground and if these points be so selected that they fall along the lines where the slopes and curvature change most abruptly the surfaces between those lines may be considered as plane surfaces and so represented on the map. Since three points determine a plane, it is best to select the points on the ground in such wise that lines joining them shall divide the surface into triangles in each of which the surface is approximately a plane surface. This is illustrated in Fig. 6.

The determined and plotted points are marked with their references. Lines joining these points divide the surface into triangles. On these lines contour points at 10 feet vertical intervals are interpolated by the method just explained. Contour points having the same reference are joined by straight lines crossing the several triangles.

In practice, the sides of the triangles are not actually drawn, except by the rows of dots made in interpolating contour points, and each contour is drawn as a continuous curved line passing through contour points of the same reference. This is shown in Fig. 7 which is constructed from the same plotted points as those used in Fig. 6, and which represents practically the same surface. The curved contours furnish a graphical and natural representation of the curved surface of the ground.

It will now be seen that in surveying the proper selection of the points to be determined on the ground is an important consideration, and that a careful study and analysis of the shape of the ground is the first requirement. The controlling agents in the formation of the earth's features are rainfall and drainage, and these elements constitute the best basis for a study of the forms which they produce. Wherever it falls, the water flows along the lines of steepest slope and increases in volume as it flows. From the highest lines of ridges and spurs it diverges and flows away on either side to lower levels. Along the lowest lines in the valleys, between ridges and spurs the water accumulates in streams and cuts channels in the soil, making gullies, arroyos, ravines, cañons or wide river bottoms between bluffs, depending on the volume of water, on the changes in volume and on the character of the soil. Erosion of the surface, caving banks, land slides on undermined bluffs, etc., gradually work the changes that result in the irregular rounded forms called ridges, knolls, shoulders, hills, spurs, cols or saddles, ravines, valleys, etc.

The highest line that can be traced on the ground between two adjacent valleys, or the line that parts the water flowing into one valley from that flowing into the adjacent valley, is called a *watershed.*

The lowest valley line along which the water from both sides accumulates and flows in a stream, is called a *water course.*

On any surface the direction of steepest slope at any point is the direction of a line tangent to the surface at that point and perpendicular to a horizontal line of the surface through that point. If the directions of steepest slope be followed from point to point in a surface, the line so traced is a line of steepest slope of the surface. At every point it is normal to the horizontal of the surface that passes through the point, and it is the line along which water will flow under the action of gravity.

On the surface of the ground the lines of steepest slope, or water-flow lines, are normal to the contours, because contours are horizontals of the surface. Watershed lines and water course lines are normal to the contours and are, therefore, lines of steepest slope, and all other lines of steepest slope join them tangentially.

The forms that have been described as resulting from the erosive action of water are not always apparent because many soils and rock resist erosion, and because other agencies are or have been at work. Frosts, winds, waves, glaciers, earthquakes and volcanoes have done their share, and may have produced the most varied and fantastic forms. As a rule, however, the drainage system of a country is the best guide in studying its topography.

The water course lines are usually plainly marked by the channel or gully washed out by the running water. Watershed lines are not so apparent, but may be traced by following the highest line of the ridge or spur. On the slopes between watersheds and water courses the lines that separate the steeper from the gentler slopes can usually be traced.

MAP READING.

As careful study of the ground is required in order that its surface may be properly determined and represented on the map, so also is careful study of a map necessary to an understanding of the information that it conveys.

Note first the scale of the map and keep in mind a map distance that represents a convenient unit of measure, such as 100 feet, 1000 yards, or 1 mile. Keep in mind also the vertical interval between contours. Note the drainage system as indicated by the water course lines and by the directions of the lines of steepest slope (normal to the contours). Trace the watersheds and note that they always terminate at the junction of two water courses.

A contour is a continuous line, and must close on itself. If it does not close within the limits of the map, it must cross the map continuously.

Contours will unite and form one line only where they represent a vertical surface, like the face of a vertical cliff.

Contours can cross only where they lie in an undercut or overhanging surface.

Eroded banks too steep to be represented by contours are usually indicated by hachures.

A single contour cannot lie between two contours both having a greater or lesser reference than its own.

Closely spaced contours indicate steep slopes, and widely spaced contours indicate gentle slopes. A wide space that contains no contours is probably a horizontal plane, or at least a surface within which the differences of elevation are less than the vertical interval between contours.

If the spacing of contours increases from the top toward the foot of a slope, the slope is concave; if the spacing increases from the bottom toward the top, the slope is convex.

A change from wide to close spacing and again to wider spacing indicates a point of inflection or of reversal of curvature at the point of closest spacing. A change from close to wide and again to closer spacing indicates a point of inflection or of reversal of curvature at the point where the spacing is widest.

A contour which forms a closed ring or oblong between two continuous contours may indicate either a knoll or a depression. If its reference is the same as that of the higher of the adjacent contours, it represents a knoll, and if the same as that of the lower, it represents a depression. Depressions usually contain water, and wherever water is shown a depression is of course evident.

A stream or water course indicates a valley, and valley contours are always convex or salient toward the head of the stream. Ridge or spur contours are convex in the direction of the downward slope.

The elevations of all points that lie on contours are given by the references of the contours. The elevation of any point not on a contour may be found by interpolating between the references of adjacent contours.

The slope of the surface at any point may be found in terms of the tangent of the angle of slope by dividing the vertical interval between contours by the scaled distance between contours at the point considered. Knowing the tangent, the angle in degrees may be taken from a table of tangents.

The point A (Fig. 8) is one-third the distance from the 300 to the 320 contour, and its elevation is therefore (307).

The vertical interval is 20 feet and the scaled distance between contours at A is 540 feet, the tangent of the slope is therefore $\frac{20}{540}$ = .037 = tangent of 2° 7′. In any other direction as long as the line c d through the point A the tangent of the slope is the vertical interval 20 feet divided by the scaled length of the intercept c d of the line between adjacent contours, and is $\frac{20}{1100}$ = .0182 which is the tangent of 1° 3′.

To trace on the surface through a given point a line of given slope, reduce the tangent of the slope to a fraction having the vertical interval as its numerator. The denominator of that fraction taken from the scale of feet will be the intercept which the required line must have between adjacent contours. Thus, if the given slope be 1° 3′, its tangent is .0183 $= \frac{20}{1100}$ Mark off 1100 feet from the scale on the edge of a strip of paper, and lay the edge through the given point A so that one mark shall fall on the 300 contour and the other on the 320 contour, as at c d, or c′ d′. The line may be continued with the same slope up to e and down to b by the same method.

Is the point C (260) visible from the point B (460) over the intervening knoll at D (320)? The point C is 200 feet below B, and the distance is 7400 feet, giving $\frac{200}{7400} = \frac{1}{37}$ as the tangent of the angle of depression from B to C. The intervening point D is 140 feet below B and the distance is 3700 feet, giving $\frac{140}{3700} = \frac{1}{26.4}$ as the tangent of the angle of depression from B to D. The point D therefore lies below the line from B to C, and C is visible.

Graphical Solution.—Lay a strip of ruled paper (cross-section or profile paper) along the line C B. Assume a convenient vertical scale and mark it on the divisions at the end of the strip, beginning with 260, the reference of the lowest contour.

A *profile* is a line cut from the surface of the ground by a vertical plane.

To construct a profile on the line C B, at each contour point, trace by eye a perpendicular on the ruled paper and mark its intersection with the corresponding horizontal as shown by the vertical scale. A line C D′ B′ drawn through the points so determined is the profile on the line C B. Draw the straight line B′ C. It passes above the knoll at D′ and C is therefore visible.

To find where a given line pierces the ground, construct a profile on the projection of the given line and on the profile draw the given line. From the point of intersection of profile and given line drop a perpendicular on the projection of the line. This will determine the point where the given line pierces the ground. Thus, a line of sight from B (460) is tangent to the knoll at D. Where does it pierce the ground? Construct the profile B′ D′ C and draw a line from B′ tangent to the knoll at D′. It intersects the profile at E′ from which drop a perpendicular to E. which is the point where the given line pierces the ground. The ground from D to E is invisible from B.

To determine areas which are invisible from an assumed point on a map, draw lines radiating from the assumed point and, with a strip of profile paper, construct on each line a profile of the ground. From the profile position of the assumed point, draw on each profile lines tangent to the salient parts of the profile, and prolong each tangent to the point where it pierces the profile. Drop perpendiculars from the profile points of tangency and from points where the tangents pierce the profile, to determine the corresponding points on the map. Draw a line on the map through the successive points of tangency. It will be a horizon line for a point of view at the assumed point. Draw a line on the map through the points where the tangent lines pierce the ground; the area between this line and the horizon line will be invisible from the assumed point.

If the tangent lines do not pierce the ground beyond the points of tangency, the horizon line determined by them is the most distant horizon, and all of the ground beyond it is invisible from the assumed point.

In drawing profiles for the solutions of problems like the foregoing, only such parts of the profile as are necessary in the solution need be drawn. An inspection of the ridge and valley forms indicated by the contours will show which parts of the profile need be drawn.

In drawing profiles the vertical scale is exaggerated, and the slopes indicated are much steeper than the natural slopes.

RELATION BETWEEN VERTICAL INTERVAL AND SCALE.

Since the vertical interval (which will be called V. I.) and the scale remain constant for any map, it is evident that the only variable for different slopes on a map is the map distance between contours (called M. D.) and that a scale can be constructed showing the M. D.'s that correspond to different degrees of slope.

Let A B (Fig. 9) represent a profile cut from the surface of the ground by a vertical plane, which also cuts A C and D B from adjacent contour planes, B C being the V. I. Then will A C be the horizontal ground distance (G. D.) between contours, to be represented on the map by the corresponding map distance, M. D. Let A be the angle of slope or the angle B A C.

Then will

$$AC = BC \text{ cotan } A$$

$$\text{or G. D.} = \text{V. I. cotan } A$$

and if the scale of the map be $\frac{1}{n}$

$$\text{M. D.} = \frac{1}{n} \times \text{G. D.} = \frac{1}{n} \times \text{V. I.} \times \cot A,$$

n being the denominator of the R. F. Find the values of M. D. for successive values of A in degrees and make them the ordinates of a curve with the corresponding values of A for abscissæ.

Thus, let n = 2000 and V. I. = 10 ft., then

$$\text{For } A = 1^\circ, \cot A = 57.3 \text{ and}$$

$$\text{M. D.} = \frac{10 \times 57.3}{2000} = 0.286 \text{ ft.} = 3.44 \text{ in.}$$

For $A = 2^\circ$,	$\cot A = 28.6$,	M. D. = 1.72 in.
$A = 3^\circ$,	$\cot A = 19.1$,	M. D. = 1.15 in.
$A = 5^\circ$,	$\cot A = 11.4$,	M. D. = 0.69 in.
$A = 7^\circ$,	$\cot A = 8.14$,	M. D. = 0.48 in.
$A = 10^\circ$,	$\cot A = 5.67$,	M. D. = 0.34 in.
$A = 15^\circ$,	$\cot A = 3.73$,	M. D. = 0.22 in.
$A = 20^\circ$,	$\cot A = 2.75$,	M. D. = 0.17 in.
$A = 30^\circ$,	$\cot A = 1.73$,	M. D. = 0.10 in.
$A = 40^\circ$,	$\cot A = 1.19$,	M. D. = 0.07 in.
$A = 60^\circ$,	$\cot A = 0.58$,	M. D. = 0.035 in.
$A = 90^\circ$,	$\cot A = 0.$,	M. D. = 0.0 in.

Lay off degrees along an axis as shown in Fig. 10, and at the degree marks erect ordinates equal to the corresponding M. D.'s. Join the points thus determined by a fair curve and it will be the curve of M. D.'s for a scale of $\frac{1}{2000}$ and V. I. = 10 feet.

For small angles the cotangent varies inversely as the angle nearly, hence, without appreciable error

$$\cot A = \frac{\cot 1^\circ}{A}$$

and the formula $M.D. = \frac{1}{n} \times V.I. \cot A$ becomes

$$M.D. = \frac{V.I. \cot 1^\circ}{n\,A}$$

in which A is the only variable. Remembering that the cot $1^\circ = 57.3$, the scale of map distances may be computed without reference to a table of cotangents.

Find the M. D. for 1° then
For 2° divide by 2
For 3° divide by 3
etc. etc.

The scale or curve thus determined is also plotted in Fig 10. The divergence of the two curves is not apparent up to 20°, and at 30° the divergence is still small. The method just given may therefore be used for determining M. D. up to 20°.

In Fig. 8 the scale is $\frac{1}{21120}$, and the V. I. is 20 feet. Substituting in the formula

$$M.D. = \frac{V.I. \cot 1^\circ}{n\,A}$$

and making $A = 1^\circ$ there results

$$M.D. = \frac{20 \times 57.3 \times 12}{21120} = 0.65 \text{ in.}$$

as the map distance between contours for a slope of 1°.

Then for 1°, M. D. $= \frac{.65}{1} = .65$ in.
for 2°, M. D. $= \frac{.65}{2} = .33$ in.
for 3°, M. D. $= \frac{.65}{3} = .22$ in.
for 4°, M. D. $= \frac{.65}{4} = .16$ in.
for 5°, M. D. $= \frac{.65}{5} = .13$ in.
for 6°, M. D. $= \frac{.65}{6} = .11$ in.
for 7°, M. D. $= \frac{.65}{7} = .09$ in.
for 8°, M. D. $= \frac{.65}{8} = .08$ in.
for 9°, M. D. $= \frac{.65}{9} = .07$ in.
for 10°, M. D. $= \frac{.65}{10} = .065$ in.
for 11°, M. D. $= \frac{.65}{11} = .06$ in.
for 12°, M. D. $= \frac{.65}{12} = .054$ in.
etc. etc.

Space the degree marks on the axis 0-12° (Fig. 8), and at the degree marks erect the corresponding ordinates equal to the M. D.'s just determined. Draw the curve as shown. Intermediate M. D.'s may be found by interpolating at the proper point of the curve. Thus for 4°-30′ take the ordinate midway between 4° and 5°.

In the formula $\text{M. D.} = \frac{\text{V. I.} \cot 1^\circ}{n\,A}$ it is evident that

V. I. and n may be multiplied or divided by the same number without changing the values of M. D. Therefore, a standard system of scales and corresponding V. I.'s may be adopted, having the same scale of M. D.'s or slopes for all maps that conform to the standard.

For military maps the system adopted is based upon military requirements, as explained in the chapter on scales, and is one in which

$$\frac{\text{V.I.}}{n} = \frac{1}{1056}$$

or is that in which the scale expressed in "inches to 1 mile" multiplied by the "V. I." is equal to 60, as shown in the following table:

Scale.		V. I. Feet.	Remarks.
Inches to 1 mi.	R. F. $= \frac{1}{n}$		
½	$\frac{1}{126720}$	120	
1	$\frac{1}{63360}$	60	Route map.
1½	$\frac{1}{42240}$	40	
2	$\frac{1}{31680}$	30	
3	$\frac{1}{21120}$	20	Road map.
4	$\frac{1}{15840}$	15	
5	$\frac{1}{12672}$	12	
6	$\frac{1}{10560}$	10	Position map.
7½	$\frac{1}{8448}$	8	
8	$\frac{1}{7920}$	7½	
10	$\frac{1}{6336}$	6	
12	$\frac{1}{5280}$	5	Fortification map.
15	$\frac{1}{4224}$	4	
20	$\frac{1}{3168}$	3	
30	$\frac{1}{2112}$	2	
60	$\frac{1}{1056}$	1	

The scales and corresponding vertical intervals found most useful for military maps are those marked "route map," "road map," "position map," and "fortification map," respectively.

The map shown in Fig. 8 conforms to this system as a position map and the scale of slopes there shown is the standard scale for this system.

The eye soon learns to estimate the map distances that correspond to different degrees of slope, and by practice great facility is acquired, not only in reading the slopes shown by contours on a map, but also in making a sketch map of the ground where slopes are often estimated and contours spaced by eye.

If no system be adopted this facility can never be acquired.

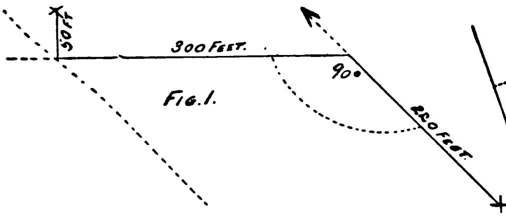


FIG. 1.

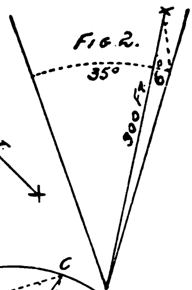


FIG. 2.

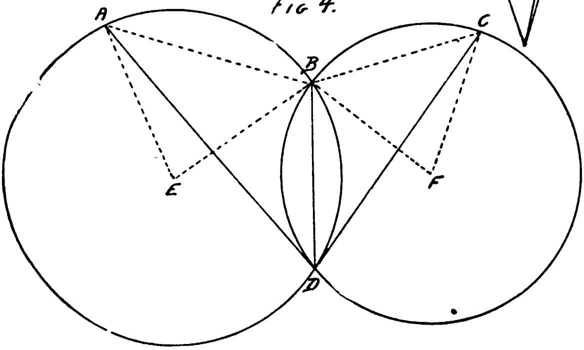


FIG 4.

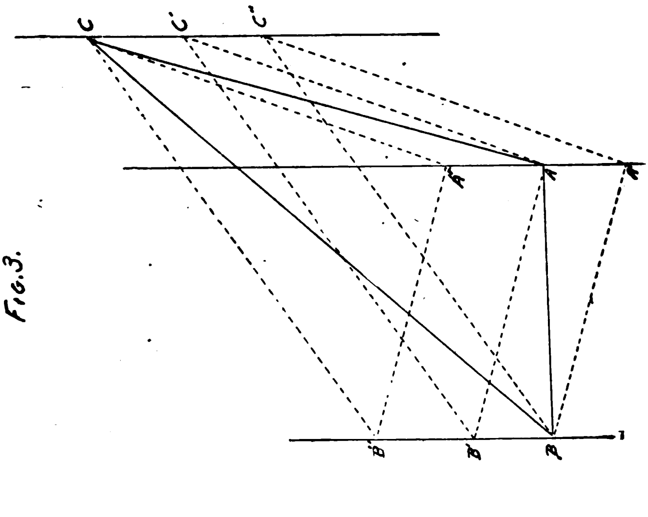
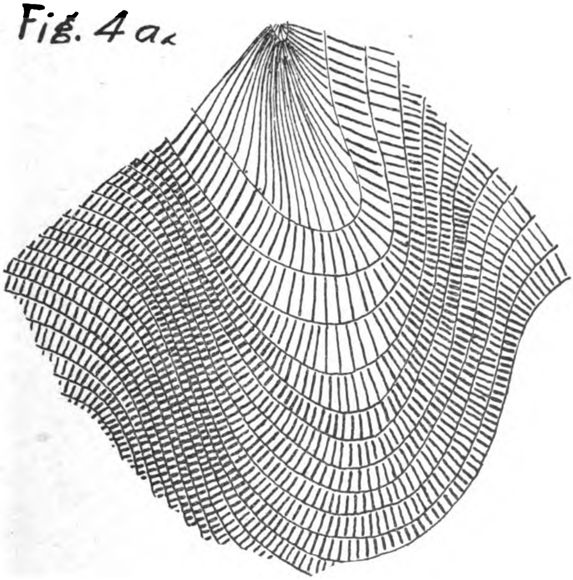


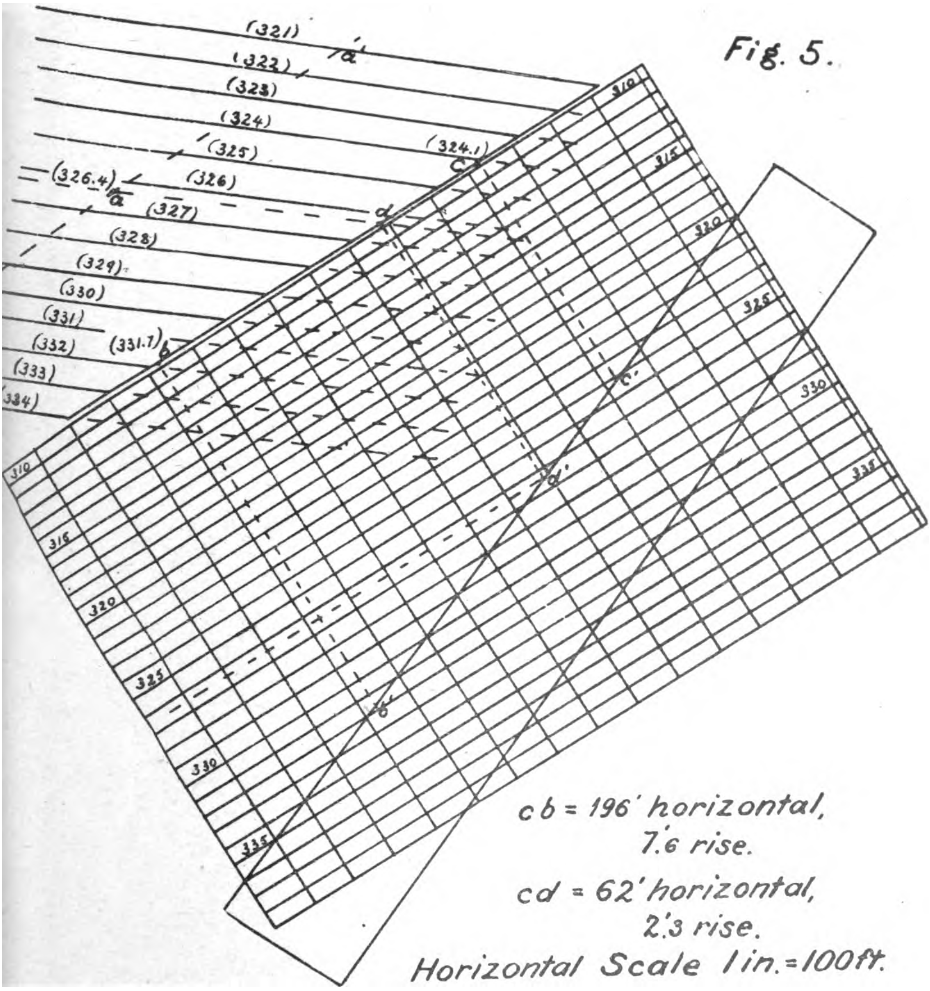
FIG. 3.

Fig. 4 a.



Scale 6" = 1 mi. $\frac{1}{10560}$ V.I. = 10

Fig. 5.

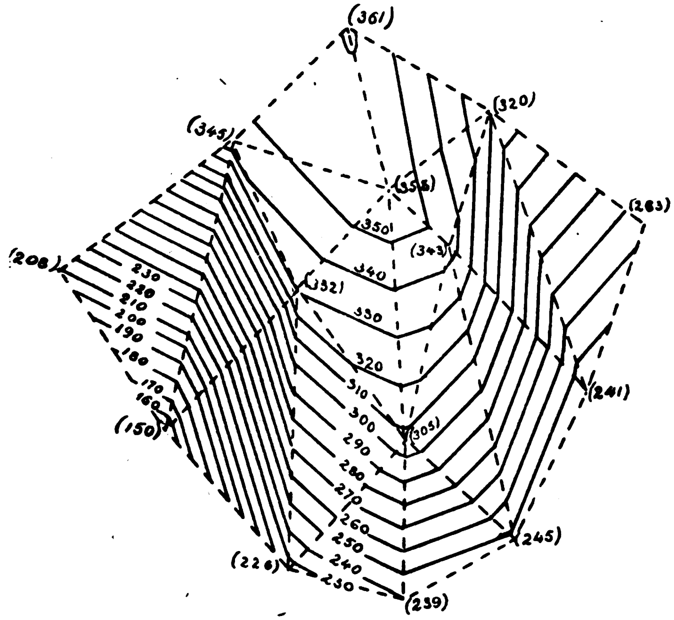


cb = 196' horizontal,
7.6 rise.

cd = 62' horizontal,
2.3 rise.

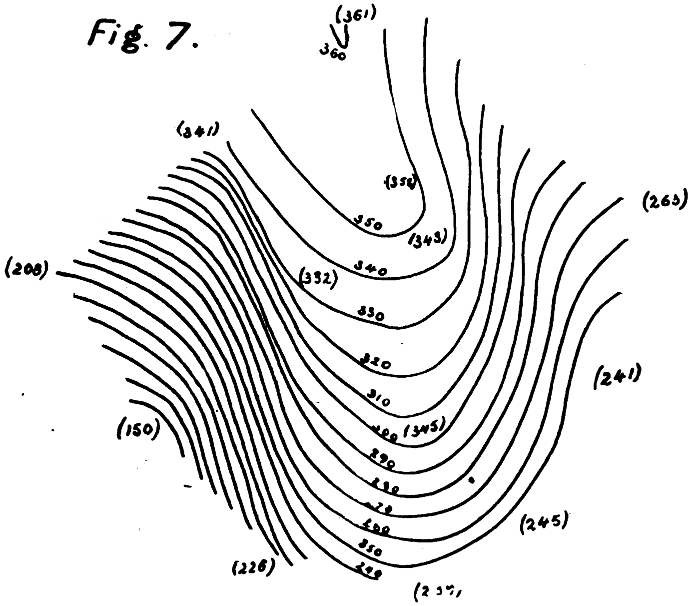
Horizontal Scale 1 in. = 100 ft.

Fig. 6.



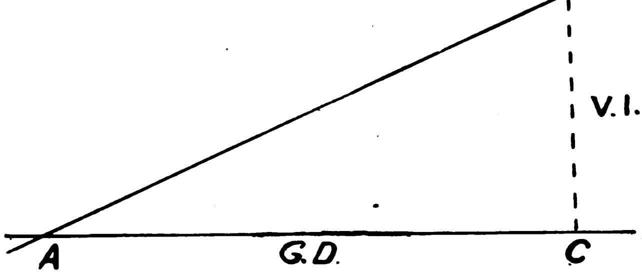
Scale 6" = 1 mi. $\frac{1}{10560}$ V.I. = 10'.

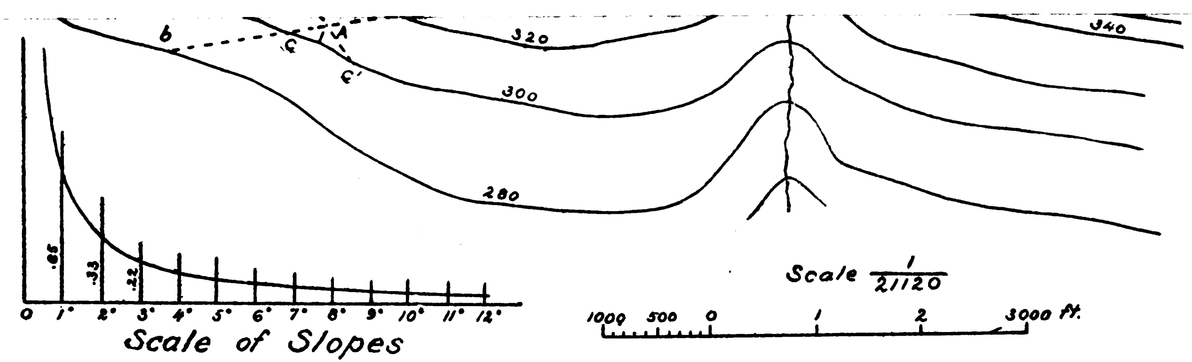
Fig. 7.



Scale 6" = 1 mi $\frac{1}{10560}$ V.I. = 10'

Fig. 9.





b
C
A
C'
320
300
280
340
.65
.33
.22
0 1° 2° 3° 4° 5° 6° 7° 8° 9° 10° 11° 12°
Scale of Slopes
Scale 1/21120
1000 500 0 1 2 3000 ft.

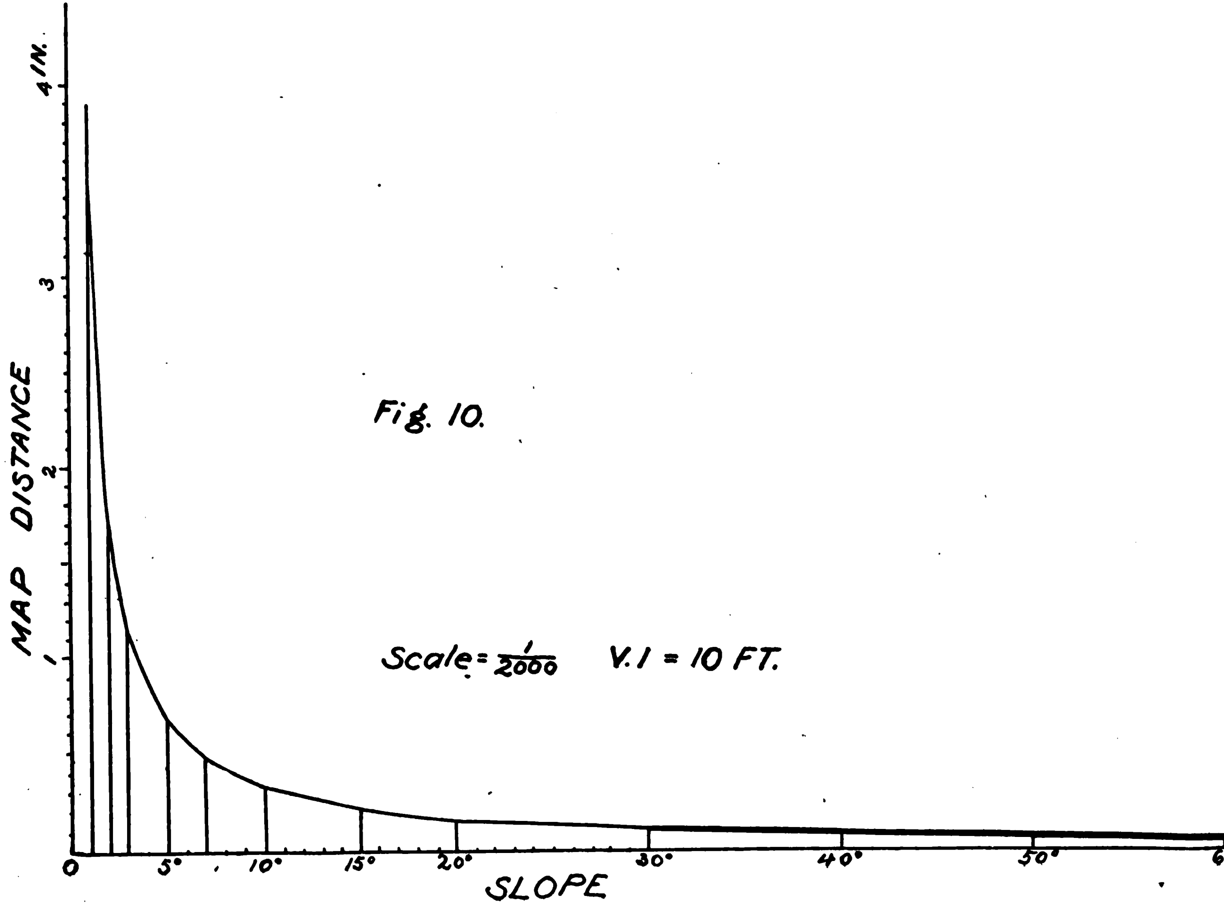


Fig. 10.

Scale = $\frac{1}{2000}$ V.I = 10 FT.

CHAPTER II.

SCALES.

The scale of a drawing is the ratio obtained by dividing the distance between two points on the drawing by the distance between the corresponding points of the object represented.

In topography the scale of a map is the ratio obtained by dividing the distance between two points on the map by the horizontal distance between the corresponding points on the ground.

This ratio or scale is in practice expressed in three ways.

1st. In the form of a fraction which expresses the above described scale ratio. This fraction is always reduced to one in which the numerator is unity and in this form it is called the representative fraction or R. F. Thus R. F. $\frac{1}{4800}$ means that any map distance is $\frac{1}{4800}$ of the corresponding horizontal ground distance, and vice versa, that the distance between two points on the ground is 4800 times the distance between the corresponding points on the map. Therefore if the numerator be taken to represent a map distance, as 1 inch or 1 foot, or 1 centimeter, the denominator will represent the corresponding ground distance in the same unit of measure. If the R. F. is $\frac{1}{4800}$, then 1 inch on the map represents 4800 inches on the ground; 1 centimeter on the map represents 4800 centimeters on the ground; 1 foot on the map represents 4800 feet on the ground, etc. The same scale ratio may be expressed in many different forms, thus

$$\frac{1}{4800},\quad \frac{1 \text{ in.}}{400 \text{ ft.}},\quad \frac{3 \text{ in.}}{1200 \text{ ft.}},\quad \frac{1 \text{ centimeter}}{48 \text{ meters}}$$

are equivalent expressions, and the transformations suggested are useful in solving problems relating to scales.

The R. F. is the universal means of comparing scales, no matter in what country and with what system of measurement the map was constructed. It is the basis of the construction of all working scales and of the solution of all problems relating to scales.

2d. The scale may be expressed in the form of a statement, in words and figures, of the relation between map and ground distances. For example, 3 inches = 1 mile, 1 inch = 200 feet.

3d. The scale may be expressed graphically by drawing a line on the map and marking its divisions not with their actual lengths, but with the ground distance that they represent. In other words, instead of writing the statement, "3 inches = 1 mile," draw a line 3 inches long and mark it 1 mile.

If a photograph could be made from a great height in a camera pointed vertically downward, the result would be a map of the region included in the picture. If it included a straight road with mile posts marked 0, 1 mi., 2 mi., 3 mi., etc., these mile posts would appear in the picture with the same marks, 1 mi., 2 mi., etc., constituting a scale of miles for the map. If the dimensions of the picture were $\frac{1}{63360}$ of the dimensions of the area covered, the scale of the map would be R. F. = $\frac{1}{63360}$ or 1 inch = 1 mile, and the actual distance between mile posts in the map or picture would be 1 inch. This is exactly what every graphical scale on a map should be, viz: a picture or reproduction on the map of distances on the ground. Such a scale furnishes a convenient method of ascertaining at once without calculation the value of any ground dimension, and is generally used in connection with either or both of the other methods. It has the additional advantage of remaining true under all conditions of enlargement or reduction by photography, or shrinking or expansion of the paper due to atmospheric influences.

Suppose we reduce a map by photography so as to halve the linear dimensions. If the original map bears the legend 1 inch = 200 feet and this is not erased or covered, it is no longer true. A building 200 feet long would be represented on the original map by 1 inch. On the reduced map the length of its representation would be but ½ inch. So, the scale would be ½ inch = 200 feet. But the inch of the original graphical scale marked 200 feet would itself be reduced to ½ inch and the scale would remain true. Like the legend, the R. F. would no longer be correct, the numerator being halved while the denominator remains the same.

THE SELECTION OF THE SCALE.

When a drawing is to be made of any object to serve a technical purpose, it must be drawn to scale.

Suppose you want to have a desk or table constructed according to your own idea. The only way in which you could make the carpenter or cabinet maker understand what you want would be by a drawing. It would be inconvenient and unnecessary to make a full sized drawing to natural scale, and you would take any sheet of paper that you might have at hand, say a double sheet of letter paper, which unfolded would be about 11x16 inches. If you are going to have your desk made about four feet long, you would draw a line say twelve inches long to represent one edge of the desk, and in doing so you have adopted a certain scale, viz:

12 inches = 4 feet:
or, 12 inches = 48 inches:
or, 1 inch = 4 inches:
or, 3 inches = 1 foot:
or, 1 foot = 4 feet:
or, R. F. = 1/4:
or, 1 centimeter = 4 centimeters:
or, .01 meter = .04 meters.

All of which are equivalent expressions and mean exactly the same thing. The expression ordinarily used would be

R. F. 1/4,
or, scale, 3 inches = 1 foot,

or the graphical expression, a line six inches long representing two feet.

Let it be required to draw the plan of a building eighty feet long by fifty feet wide, and to show doorways, window openings, thickness of walls and partitions, and similar details. The first question that arises is, "What scale shall be adopted?" The governing conditions are that the smallest details required must be shown to scale, and that the sheet or drawing must not be so large as to be unwieldy and inconvenient. The smallest detail to be represented is the thickness of partitions, say nine inches. The largest is the length of the building, eighty feet. For the supposed purpose of the drawing it will suffice to be able to scale off dimensions to the nearest $\frac{1}{4}$ inch, and if $\frac{1}{100}$ inch be assumed as the smallest portion of a line clearly appreciable to the eye, we should use a scale not smaller than $\frac{1}{100}$ inch $= \frac{1}{4}$ inch, or 1 inch $= 25$ inches; or R. F. $\frac{1}{25}$. Now what will be the dimensions of the drawing if this scale is adopted? The length of the drawing will be $\frac{1}{25}$ of the length of the building. $\frac{1}{25} \times 80$ feet. $= 38\frac{2}{5}$ inches, and the width $\frac{1}{25} \times 50$ feet $= 24$ inches. These are less than the length and width of one of the ordinary sizes of sheets of drawing paper, which are 27×40 inches, and the scale adopted would be suitable.

If the smallest details required were the widths of doors and windows to the nearest inch, we would take $\frac{1}{100}$ inch $= 1$ inch, or 1 inch $= 100$ inches, or a scale of R. F. $\frac{1}{100}$, or 1 inch $= 8\frac{1}{3}$ feet. For convenience take 1 inch $= 8$ feet, and the drawing would be only 10 inches long and $6\frac{1}{4}$ inches wide.

Now pass to the consideration of a larger subject, and assume that a representation of the Post of Fort Leavenworth is required, including only the portion occupied by buildings. Here again the first consideration is the scale to be adopted, as determined by the purpose or object which the map is to subserve. On the one hand the map might be intended to show to the nearest foot the dimensions of all buildings and the distance between them, the widths of roads and sidewalks, and the location of water and gas pipes, sewers, hydrants, fences, small streams, bridges, culverts, etc. On the other hand, it might only be required to show the general shape of buildings and their relative location, the position and average width of roads and walks, the car lines, if any, and perhaps the water mains and principal sewers, with an allowable limit of error in plotting or scaling of about four feet.

A map must frequently be used and referred to in the field where there is no table to spread it upon, and no dividers available for taking off distances accurately. For this reason $\frac{1}{50}$ inch is considered the smallest portion of a line that can readily be estimated or scaled on maps in the field. Then in the first case cited above, we should take not less than $\frac{1}{50}$ inch to represent 1 foot or 12 inches, and we have a scale of

$$\tfrac{1}{50}\text{ in.} = 12\text{ in.}$$

$$\text{or, } 1\text{ in.} = 600\text{ in.} = 50\text{ ft.}$$

$$\text{or, R. F.} = \tfrac{1}{600}.$$

The principal post buildings are included within a space say 3000 ft. long by 2400 ft. wide, and the map dimensions would be $\frac{1}{600}$ of these respectively, or five feet long by four feet wide. This is not an excessive size, and the scale adopted would be suitable for the purpose. If necessary, however, the drawing could be made on two sheets of a more convenient size.

For the second case cited we would take $\frac{1}{50}$ inch to represent four feet, or 1 in. = 200 ft. = 2400 in., or R. F. = $\frac{1}{2400}$. The size of the map would be $\frac{1}{2400} \times$ 3000 ft. = 15 in. long and $\frac{1}{2400} \times$ 2400 feet. = 12 in. wide.

In the same manner we would determine the scale suitable for a map of the Reservation of Fort Leavenworth. This reservation is, we will say, four and one-quarter miles long and three miles wide in the extreme dimensions. The smallest details required would be the buildings, and these not in accurate dimensions, but simply in their relative positions and approximately true in size and general form. A possible error in plotting and scaling of ten feet would be allowable, and we will therefore test a scale of $\frac{1}{50}$ inch = 10 ft., or 1 in. = 500 ft. = 6000 in., hence R. F. = $\frac{1}{6000}$.

The longest dimension on the ground is $4\frac{1}{4}$ miles $\times$ 5280 = 22440 ft., and the length of the map will be $\frac{1}{6000} \times 22440$ ft. = 3.74 ft. = 44.88 in., or say 45 in., width 31.68 in. Drawing paper comes in rolls and in sheets. The rolls are from 30 in. to 62 in. wide, and 10 to 40 yds. long. We can, therefore, cut from the rolls any size of sheet that is desired. The sheets vary in size from "Cap" 13 in. x 17 in., to Antiquarian, 31 in. x 53 in. A convenient size is the Double Elephant, 27 in. x 40 in. If this last is the size of sheet available for the map now under consideration, the scale tested is found to be too large, and the question arises, what reduction of scale is necessary. The drawing must be reduced from 45 in. to 36 in. in length to provide for a border 2 in. wide at the ends. The ratio of reduction is, therefore, $\frac{36}{45} = \frac{4}{5}$, and the new scale will be $\frac{1}{6000} \times \frac{4}{5} = \frac{1}{7500}$: $\frac{1}{7500} \times 22440$ ft. = 3 ft. = 36 in. long. $\frac{1}{7500} \times 15840$ ft. = 2.112 ft. = $25\frac{1}{3}$ in. wide, which leaves nearly one inch border at top and bottom.

We have now seen that the first consideration in determining the scale for a drawing is the accuracy with which the smaller details are to be delineated, and that the second consideration is the size of the resulting map or drawing.

There is a third consideration not yet mentioned, viz: the degree of precision of the instruments or the means used to measure the distances and directions on the ground. If a triangulation survey is to be made with an accuracy of $\frac{1}{10000}$ in distances and to the nearest 25″ of arc in direction, the probable error of measurement for the average length of lines (say three miles) would be about 30 inches. The scale then should provide for the plotting and scaling of distances to the nearest 30 inches. In the extreme care and precision that should be used in plotting the triangulation stations of an accurate survey, $\frac{1}{200}$ of an inch on the paper would not be overlooked, and the resulting scale is

$$\frac{1}{200} \text{ in.} = 30 \text{ in.}$$
$$\text{or, } 1 \text{ in.} = 6000 \text{ in.} = 500 \text{ ft.}$$
$$\text{R. F.} = \frac{1}{6000}, \text{ and}$$
$$10.56 \text{ in.} = 1 \text{ mi.}$$

Now consider, on the other hand, a foot reconnaissance made by pacing distance and using the hand compass for directions. The average length of courses may be assumed as 2000 feet. An accuracy greater than $\frac{1}{100}$ can not be expected, and the probable error in the average course would be $\frac{1}{100} \times 2000$ or 20 feet. It would be absurd to use a scale by which five feet could be clearly represented, when the actual error in the measured distance may be 20 feet. Neither do we wish to increase the chances of error by using too small a scale. To keep on the safe side let us take $\frac{1}{50}$ of an inch to represent 18 feet; then

$$1 \text{ inch} = 900 \text{ feet} = 10800 \text{ inches,}$$
$$\text{R. F.} = \frac{1}{10800}$$
$$5.89 \text{ inches} = \text{a mile,}$$

or, for convenience, say

$$6 \text{ inches} = 1 \text{ mile,}$$
$$\text{with R. F.} = \frac{1}{10560}$$
$$\text{and } 1 \text{ inch} = 880 \text{ feet.}$$

This is the largest scale that one would be justified in using under the circumstances, and is suitable for the sketch of a limited area in which all the details and accidents of the surface are to be shown, as in the case of a military position to be occupied for defensive purposes, or of a site for the encampment of an army.

SCALES FOR SKETCHING.

The first subject that presents itself in connection with military field sketching is the question of the scale, and this must be considered with reference to the purpose of the sketch and the amount of detail that the sketch must show in order to fulfill its purpose.

The two general classes of sketches are Road Sketches and Position Sketches.

The first are made with a view to organizing and ordering the marches of any army, and to this end the sketch should show distances by the scale to the nearest minute of marching—say 80 yards, in each hour's march; it should show halting places, passing places and camping sites; in towns and villages it should show the principal intersecting streets, perhaps only 100 yards apart; to properly develop the slopes and forms of the ground the sketch should show contours at 20 ft. vertical interval without crowding them or confusing other details; in practicable country the slopes will not often exceed six degrees, which will place the contour lines about 60 yards apart. On the map the contours should not lie closer than $\frac{1}{10}$ of an inch apart. As this is the smallest detail to be considered, it should be used as a test for determining the scale and their results $\frac{1}{10}$ in. = 60 yds., or $1'' = 21600''$, which is practically $3'' = 1$ mi.

This scale will show all other necessary details with the requisite accuracy, and it will place a day's mounted reconnaissance of say fifteen miles on a paper 45 inches long, a convenient length for the sketching case, or a day's foot reconnaissance of ten miles on two lengths of the 15-inch drawing board.

This scale, $3'' = 1$ mile, R. F. $\frac{1}{21120}$ is adopted for all road sketches.

For position sketches the primary considerations are the posting of troops in position down to regiments or even battalions, the scaling of ranges to the nearest 15 yards, and the definition of slopes and forms of the surface of the ground by contours at not more than 10 feet vertical interval.

For scaling ranges, the scale should be not smaller than $\frac{1}{20}$ in. = 15 yards., or $\frac{1}{10800}$, or about $6'' = 1$ mi.

For posting troops, the general should be able to indicate on the map, to his colonels, by a line about one inch long, the front to be occupied by each of his regiments, say 300 yards, which requires a scale of $\frac{1}{10800}$ or about $6'' = 1$ mi.

For contours of 10 ft. V. I. and slopes up to 6° the smallest horizontal distance between contours will be about 30 yds. To keep the contours for these slopes at least $\frac{1}{10}$ in. apart on the map requires $\frac{1}{10}$ in. = 30 yds. or $\frac{1}{10800}$ or about $6'' = 1$ mi.

For all the purposes of a position sketch, therefore, a scale of 6 in. to the mile will suffice, and this scale is adopted.

An outpost sketch should fulfill practically the same conditions as a position sketch, and the same scale should be used.

The purposes of the road sketch may be extended to furnish, by the combination of sketches on several roads, a general map of the country to be used for organizing and ordering the march of an army on several parallel roads, for locating the cavalry screen, for arranging the details of concentrations and deployments, pursuits and retreats. For these and other purposes in grand tactics the scale of the road sketch is too large, giving maps of unwieldy size and showing much unnecessary detail. The combined road sketches will therefore be reduced to give maps of convenient size showing the immediate zone of operations say 30 to 40 miles on the sides; and for this a scale of 1 in. to the mile or $\frac{1}{63360}$, with contours at 60 ft. vertical interval is suitable.

For strategical maps showing the entire theater of operations, the published civil maps on scales of 10 to 25 miles to the inch will serve, or the tactical maps may be still further combined and reduced.

For the defensive organization of a place, which in the eventualities of war becomes important, and which must hastily be fortified by strong field works to resist a threatened attack, maps must be prepared in great detail, to show all the minor folds and accidents of the ground, all obstacles and screens, dead spaces, lines of trenches, covered communications, the field works themselves in approximate true dimensions, and accurate ranges to well defined land marks within the zone of defense. In this connection questions of defilade will arise, which gives importance to differences of elevation equal to the height of a man.

The above details can be shown on a scale of 12 in. = 1 mi., or R. F. $\frac{1}{5280}$ with contour intervals of 5 feet. A lunette 150 yards long will be represented by one inch on the map, and a map error of $\frac{1}{20}$ in. in scaling ranges would be only 7½ yards. If time were available an instrumental survey would be made, but generally the methods of field sketching would have to be resorted to for filling in, using especial care to secure accuracy.

The resulting series of scales for military field maps is as follows:

No.	R. F.	Inches to One Mile.	Contour Interval Feet.	PURPOSES.
1	$\frac{1}{5280}$	12	5	Defensive organization of an important place.
2	$\frac{1}{10560}$	6	10	Position or battle map.
3	$\frac{1}{21120}$	3	20	Road maps for marches and minor tactics.
4	$\frac{1}{63360}$	1	60	Route maps, maneuvers in grand tactics.
5	$\frac{1}{633600}$	$\frac{1}{10}$		Strategical maps.
	$\frac{1}{1584000}$	$\frac{1}{25}$		

It must not be understood that the foregoing conclusions are absolute and definite. On the contrary, for any one of the purposes considered a considerable range in the selection of a scale is permissible, and the above determinations are based as much on the past experience of many officers, as to suitability and convenience of scales, as they are on the discussions given.

But it is of the utmost importance in military work that a uniform system of scales be adopted and adhered to by all persons engaged in field sketching and in preparing and combining the resulting maps. For the purposes stated the foregoing system of scales has been adopted for military maps. For special purposes and under different conditions, the scale should be selected in accordance with the principles that have been discussed.

CONSTRUCTION OF SCALES.

In order that a graphical scale may be read quickly and conveniently it must show exact whole numbers of units, tens, hundreds or thousands of the unit of measure, and the primary and secondary divisions of the scale should also read in exact units or multiples of ten of the unit of measure.

Graphical scales are of two kinds, reading scales and working scales. A reading scale is a scale drawn on a finished map or drawing and used to read distances thereon in the desired unit of measure, such as feet, yards, meters, miles, kilometers, etc. A working scale is a scale used in making or plotting the map or drawing, and it gives distances in the unit of measure that was used in determining distances on the ground. The units of measure that may be so used are feet, yards, meters, chains, paces; steps or strides of a horse, walking, trotting or cantering; minutes (meaning the distance passed over each minute) for a horse walking, trotting or cantering; revolutions of a wagon wheel (meaning the distance passed over by the wagon for each revolution of the measured wheel); the length of a piece of cord or rope that may be used to measure distances, etc. In fact, any fixed length may be taken as a unit of measure for determining distances on the ground, provided that length be known in inches. It would be possible to reduce each measured distance, no matter what the unit used, to feet or yards, and then lay off the distance by a scale of feet or yards, but this requires a computation for each measurement, and involves time and labor. By using a properly constructed working scale reading in the units of measure used, all measured distances may be plotted directly without reduction.

In particular cases the reading scale and the working scale will be the same, as for instance when ground measurements are made in yards and the map scale is to be read in yards. Generally, however, the working scale will have a unit of measure different from that desired for the reading scale.

The reading scale is placed on the map wherever there is space available for it. Usually it appears just below the title, and questions of symmetry determine its length at about 3 or 4 inches on small maps up to about 6 or 8 inches on large maps. Its exact length will of course be determined by the total number of units of measure to be represented.

For very accurate work the working scale should be constructed on the paper in order that expansion and contraction wili affect paper and scale equally. But for ordinary work it is more convenient to construct the working scale on the edge of a ruler and apply it directly to the line to be plotted. Its length should be such that the longest measured distance may be plotted by one application of the scale.

When the scale is given as so many inches to the mile, or as one inch equals so many tens, or hundreds of feet, yards, etc., the construction of the scale of miles, feet or yards consists simply in laying off inches on a straight line and marking the points of division with the proper number of miles, feet, yards, etc., as the case may be. In other cases a simple computation and construction are necessary.

The problem may be stated as follows: Given the R. F. and the number of inches in the unit of measure, to construct the scale. When the R. F. and the length in inches of the unit of measure are not stated they must be deduced from given data.

Rule:—Transform the R. F. to a fractional form in which the denominator shall be the desired whole number of units, tens, hundreds or thousands of the given unit of measure. The new numerator will be the length of the scale in inches. Lay off this length on a straight line and divide it by construction into a convenient number of equal parts, each representing an entire number of units, tens, or hundreds, etc., of the unit of measure. Mark the points of division with the number of units of measure represented, starting from the assumed zero point.

Examples:

1. A man takes 238 paces in a distance of 595 feet. Construct a working scale of paces with a scale of 3 inches to 1 mile.

$$\frac{3 \text{ inches}}{1 \text{ mile}} = \frac{3 \text{ inches}}{63360 \text{ inches}} = \frac{1}{21120} = \text{R. F.}$$

$$\frac{595 \text{ ft.}}{238 \text{ paces}} = 2\tfrac{1}{2} \text{ ft.} = 30 \text{ inches} = 1 \text{ pace.}$$

$$\frac{1 \text{ in.}}{21120 \text{ in.}} = \frac{1 \text{ in.}}{704 \text{ paces}} = \frac{2.84 \text{ inches}}{2000 \text{ paces}}$$

Solution :—First, find as shown above the

R. F. $= \frac{1}{21120}$

Second, find the number of inches in the unit of measure; in this case, 1 pace = 30 inches.

Third, the R. F. shows that 1 inch represents 21,120 inches. Reducing the denominator to paces there results the equivalent ratio, 1 inch to 704 paces. Assume that the space available or suitable for the scale is about 3 inches long, and it is evident that 2000 paces can be represented within the available space. Using 2000 paces as the denominator of the transformed ratio, the numerator is found to be 2.84 inches.

Draw a line 2.84 inches long to represent 2000 paces (see Fig. No. 1.) and divide it into four primary divisions each representing 500 paces. Subdivide the left hand division into 10 equal parts, each representing 50 paces and mark the divisions as shown.

If the space available for the scale had been about 4 inches long the scale could be drawn to represent 2800 paces, thus:

$$\frac{1}{21120} = \frac{1 \text{ in.}}{704 \text{ paces}} = \frac{3.98 \text{ in.}}{2800 \text{ paces}}$$

that is, a line 3.98 in. long would represent 2800 paces. If divided into seven parts, each part would represent 400 paces. The left hand division would be subdivided into eight parts, each representing 50 paces.

2. With the same R. F. ($\frac{1}{21120}$) construct a working scale to read minutes for a horse that walks a mile in 15 minutes.

The R. F. being given, first find the number of inches in the unit of measure (1 minute).

$$\frac{1 \text{ mile}}{15 \text{ min.}} = \frac{63360 \text{ in.}}{15 \text{ min.}} = 4224 \text{ in.} = 1 \text{ min.}$$

Next, transform the R. F. thus:

$$\frac{1}{21120} = \frac{1 \text{ in.}}{5 \text{ min.}} = \frac{3 \text{ in.}}{15 \text{ min.}}$$

Assuming as before that the space suitable for the scale is about 3 inches long, it is found that 15 minutes will be represented by 3 inches, and the scale is so constructed (see Fig. No. 2).

3. With the same R. F. ($\frac{1}{21120}$) construct a reading scale of yards. 1 yard = 30 inches.

$$\frac{1}{21120} = \frac{1 \text{ in.}}{586.6 \text{ yds.}} = \frac{2.73 \text{ in.}}{1600 \text{ yds.}}$$

Finding that 1 inch represents 586.6 yds., it is evident that within the 3 inch space available, 1600 yds. may readily be represented, and using this as the new denominator, there results 2.73 inches as the new numerator or length of scale. (See Fig. No. 3.)

The necessary computation may be slightly simplified by the method of construction illustrated in Fig. No. 5, thus:

$$\frac{1}{21120} = \frac{1 \text{ in.}}{586.66 \text{ yds.}} = \frac{3 \text{ in.}}{1760 \text{ yds.}}$$

Lay off a horizontal line 3 inches long, and on the inclined line, with a convenient scale of equal parts, (in this case the scale of 50ths on the triangular scale) mark the points 500, 1000, 1500 and 1760, or mark each 100 if desired. Join the 1760 point with the end of the 3 inch line and draw parallel lines as shown; erase that part of the scale to the right of the 1500, or extend the scale to 2000 yards if desired. This method is applicable in all cases, but it requires a set of conveniently divided scales of equal parts like those on the triangular scale, and this is not always at hand. The first method may be used with any scale of inches, because the inch may be readily divided into tenths and distances laid off by estimation to the nearest 100th of an inch, which is sufficiently accurate for ordinary purposes.

If it is desired to scale off with accuracy distances smaller than those shown by the secondary divisions of a scale, a diagonal scale is used. The construction and use of such a scale are illustrated in Fig. No. 4. The scale on the bottom line is made in the usual manner with secondary divisions reading to 10 feet. By using the diagonal lines the scale may be read accurately to 1 foot, and by estimation between the horizontals to the nearest $\frac{1}{10}$ of a foot. The horizontal lines must be equally spaced and the diagonals must be so inclined as to cover one secondary division exactly.

In the example given in Fig. No. 4, to scale off any distance, as 273 feet, place the dividers on the horizontal line marked 3 and include the distance A B which is made up as follows:

Two primary divisions	=	200 ft.
Seven secondary divisions	=	70 ft.
$\frac{3}{10}$ of a secondary division	=	3 ft.
Total, - - -		273 ft

The diagonal scale is especially useful for reading different units on the same scale; thus, if the primary divisions read yards, the secondary divisions may be made to read feet, and the diagonal scale with 12 spaces will read inches; or, if the primary divisions reads meters, the secondary divisions may read decimeters, and the diagonal scale, centimeters; then by estimation between the horizontals the scale could be read to the nearest millimeter.

4. Construct a scale to read paces of 32 inches,

R. F. $\frac{1}{3600}$

$$\text{R. F.} = \frac{1}{3600} = \frac{1 \text{ in.}}{112.5 \text{ paces}} = \frac{6.22 \text{ in.}}{700 \text{ paces}}$$

5. Construct a scale to plot distances from time, the measurements being made by timing a horse walking a mile in 12 minutes, with R. F. $\frac{1}{20000}$. The scale will read minutes, the word in this case signifying the distance covered in a minute.

12 minutes = 1 mile = 63360 inches.
1 minute = 5280 inches.

$$\text{R. F.} = \frac{1}{20000} = \frac{1 \text{ in}}{3.78 \text{ min.}} \doteq \frac{2.91 \text{ in.}}{11 \text{ min.}}$$

6. With R. F. $\frac{1}{8000}$ construct a scale to plot measurements taken in meters, the meter being 39.37 inches.

$$\text{R. F.} \frac{1}{8000} = \frac{1 \text{ in.}}{203.2 \text{ met.}} = \frac{4.92 \text{ in.}}{1000 \text{ met.}}$$

7. Construct a scale of yards considering that 3 inches = 1 mile.

$$\frac{3 \text{ in.}}{1 \text{ mi.}} = \frac{3 \text{ in.}}{63360 \text{ in.}} = \text{R. F.} \frac{1}{21120}$$

Then with R. F. $\frac{1}{21120}$ construct scale to read yards.

8. A French map has a graphical scale only. We find that a space marked 100 meters measures 2.23 inches. Construct a scale to read yards. A meter is 39.37 inches.

$$\frac{2.23 \text{ in.}}{100 \text{ met.}} = \frac{2.23 \text{ in.}}{3937 \text{ in}} = \text{R. F.} \frac{1}{1765.46}$$

Then with this R. F. construct scale of yards.

9. A map has no scale. A certain distance which measures on the map 2.5 inches, measures on the ground 523 paces of 30 inches each. Construct scale of feet.

$$\frac{2.5 \text{ in.}}{523 \text{ paces}} = \frac{2.5 \text{ in.}}{15690 \text{ in.}} = \text{R. F.} \frac{1}{6276}$$

Then with this R. F. construct scale of feet.

10. A map has been roughly made in the field as follows: A picket rope of unknown length was used to take the measurements, and a scale was made by assuming a certain line on the paper, dividing it into equal parts and marking each division one rope. After obtaining facilities for measurement it was found that a division on the paper measured .28 inches and that the rope was 84 feet long. Construct scale of feet.

$$\frac{.28 \text{ in}}{84 \text{ ft.}} = \frac{.28 \text{ in}}{1008 \text{ in.}} = \text{R. F.} \frac{1}{3600}$$

With this R. F. construct scale of feet.

11. A map has been plotted using a scale of paces, R. F. $\frac{1}{12000}$. This scale was constructed under the supposition that a pace measured 32 inches, but it was afterward found that the pace was only 30 inches. Find the true R. F. of the map.

R. F. $= \frac{1}{12000}$. That is, one pace on the map represents 12,000 paces on the ground. A pace on the map has been plotted as equal to 32 inches, while the pace on the ground is really but 30 inches. Hence we have

$$\frac{1 \times 32}{12000 \times 30} = \frac{1}{11255}$$

which is the true R. F. of the map as constructed.

In place of the rule previously given, the following formula may sometimes be found useful in the construction of scales.

Let x = required length of scale line in inches

$\frac{1}{n}$ = representative fraction.

m = length of unit of measure in inches.

s = total number of units to be represented by the scale.

Then

$$x = \frac{m\,s}{n}$$

Assume for s a convenient round number in tens, hundreds or thousands, etc., and find the value of x to two decimal places.

PROBLEMS.

1. Construct a scale of R. F. $\frac{1}{5000}$ to read feet.

2. Construct a scale of R. F. $\frac{1}{5280}$ to read yards.

3. Construct a scale of 15 inches = 1 mile to read yards.

4. Construct a scale of 6 inches = 80 chains to read meters. One chain = 792 inches: 1 meter = 39.37 inches.

5. Construct a diagonal scale of R. F. $\frac{1}{60}$ to read yards, feet and inches.

6. A horse takes 600 steps in walking 500 yards. Construct a scale of 1 inch = 600 feet to read steps.

7. A horse trots a mile in 6½ minutes. Construct a scale of R. F. $\frac{1}{21120}$ to read minutes.

8. A man on a bicycle finds that his cyclometer registers 3½ miles when he has been going at uniform rate for 18 minutes. Construct a scale of R. F. $\frac{1}{15840}$ to read minutes.

9. Construct a scale of R. F. $\frac{1}{21120}$ to read revolutions of a 20-inch wheel.

10. A map has no scale. It is found that a distance which measures 4 inches on a map measures 1760 yards on the ground. Construct a scale of yards.

11. A French map has only a graphical scale and we have no metric scale. A length of scale marked 200 meters measures .75 inches. Construct a scale of feet. Meter = 39.37 inches.

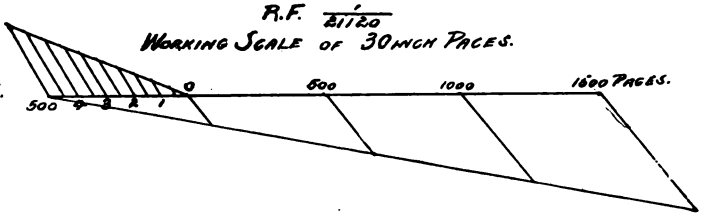
12. We wish to send a map made in this country to a place where the metric system is used. It has on it only the graphical working scale, and that shows that a line representing 2000 paces of 30 inches equals 2.84 inches. Construct a scale of meters.

13. A road sketch was made with a horse that was supposed to trot a mile in $7\frac{1}{2}$ minutes, and was plotted to a scale of R. F. $\frac{1}{21120}$. It was afterwards found that the horse trotted a mile in 7 minutes. What is the true R. F. of the sketch? Construct scale of yards.

R.F. $\frac{1}{21120}$

WORKING SCALE OF 30 INCH PACES.

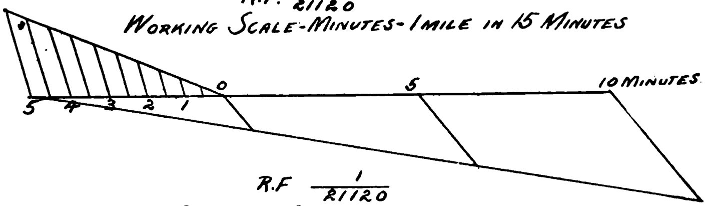
No. 1.



R.F. $\frac{1}{21120}$

WORKING SCALE—MINUTES—1 MILE IN 15 MINUTES

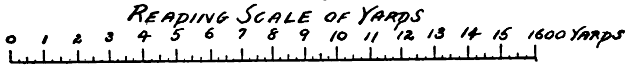
No 2.



R.F. $\frac{1}{21120}$

READING SCALE OF YARDS

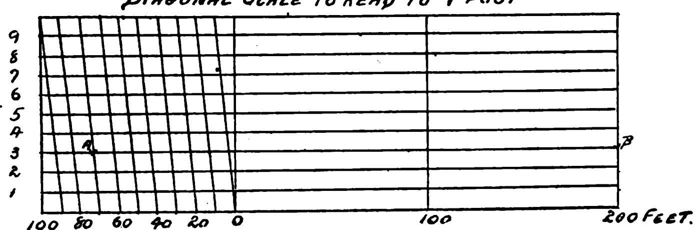
No 3



R.F. $\frac{1}{1200}$

DIAGONAL SCALE TO READ TO 1 FOOT

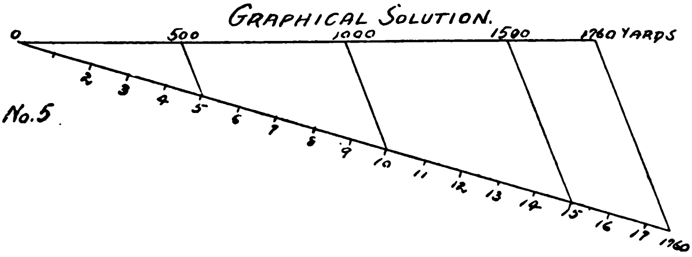
No. 4.



R.F. $\frac{1}{21120}$.

GRAPHICAL SOLUTION.

No. 5.



CHAPTER III.

INSTRUMENTS.

Surveying involves measurements of direction and distance, and the instruments used are specially designed and constructed with these objects in view.

MEASUREMENT OF DIRECTION.

The direction of one point from another is the direction of the straight line joining the two points. That direction must be determined with reference to the direction of some line that is known or that is assumed as the standard of direction. If the direction of one line be known, and the difference of direction of this line and another be determined the direction of the other becomes known.

The difference of direction of two lines is an angle, and the directions of lines are therefore determined by measuring angles. The unit of angular measurement is the degree or $\frac{1}{360}$ of a circle. To measure smaller parts of an arc the degree is divided into 60 minutes and the minute into 60 seconds.

To determine the direction of a line, that line must be physically established and must be apparent to the senses. We can imagine a straight line joining any two points, but we cannot measure an imaginary line. Nature's nearest approximation to a physical straight line is a ray of light, and this taken in its opposite sense as a line of sight is almost always used to establish a straight line between two points. It is not perfectly straight, because a ray of light, in passing through a medium like air of varying density, is by refraction deviated from its straight course. This deviation, for distances involved in ordinary surveying, is slight and may generally be neglected. Moreover, it occurs only in the vertical plane and therefore affects only vertical angles. In horizontal angles no appreciable error arises from refraction. The only other straight line that nature provides is the plumb line, and this determines a vertical line only.

To determine a horizontal line, the spirit level is used. A horizontal line or a vertical line is taken as the standard direction from which to measure vertical angles.

A horizontal line directed toward the north is usually taken as the standard direction from which to measure horizontal angles, although any known direction line may be so used.

All instruments that determine direction are so constructed as to measure the angle between the line of sight to the required point and some other known or assumed direction line.

The essential parts or elements of an instrument for measuring angles are:

1. Pointers or sights or telescope, establishing a well defined line of the instrument that may be brought, by revolution about a fixed axis, into coincidence with any desired line of sight or direction line.

2. A circular arc graduated or divided into degrees, with its center in the axis of revolution and its plane perpendicular thereto.

3. Means of placing the plane of the circular arc in the plane of the angle to be measured or its axis perpendicular to that plane.

4. An index by means of which the pointing of the sights or telescope may be read in degrees on the circular arc.

5. A case or frame or standard to connect and support the several parts of the instrument while permitting the necessary relative movements of the parts.

Every instrument used to measure angles will be found to have all of the above mentioned parts or elements.

THE TRANSIT.

The transit is an instrument designed to measure the angular coördinates, both horizontal and vertical, in any system of coördinates. With the stadia attachment it also measures the distance coördinate or radius vector.

This instrument appears in different forms, depending on the different special purposes for which designed, as the surveyor's transit, engineer's transit, mountain transit, miner's transit, etc., but they are all essentially the same in principle and operation, and differ only in size or in some special attachment.

Since the transit is designed to measure both horizontal and vertical angles it must have the parts necessary for both purposes, viz:

1. A telescope in which the line of collimation is the particular line of the instrument that is brought into coincidence with the line of sight through the known point or origin and the distant required point.

One point of this line of collimation must remain fixed at the origin while the line itself must revolve about it.

2. A horizontal axis through the fixed point perpendicular to the line of collimation, for turning off vertical angles.

3. A vertical axis through the same fixed point for turning off horizontal angles.

4. A graduated circular arc centered on the horizontal axis and perpendicular thereto for measuring vertical angles.

5. A graduated circular arc centered on the vertical axis and perpendicular thereto for measuring horizontal angles

6. Indices, which in the transit take the form of verniers for reading the angles on the vertical and horizontal circles.

7. Adjusting screws, by means of which the vertical axis may be made truly vertical; the horizontal axis, truly horizontal; and the line of collimation, perpendicular to the horizontal axis.

8. Level tubes, by means of which the verticality of the vertical axis may be assured.

9. A support for the instrument in the form of a tripod or a trivet.

THE TELESCOPE.

The parts of the telescope are the object glass, the eye piece, the cross wires, and the tube. The object glass is a compond achromatic lens. The rays of light, emanating from any point in front of the lens and passing through it, are brought to a focus behind the lens, and thus an image if the point is formed. The ray which passes through the center of the lens is not deviated, therefore the image of the point will lie on the straight line through the point and the center of the lens. Every point of an object in front of the lens will have its corresponding image behind the lens, and thus an image of the object is formed which may be viewed by means of the eyepiece. Each point of the object, the corresponding image of the point, and the center of the lens, lie in the same straight line, therefore the image of the object is reversed, or appears as if revolved 180 degrees about the axis of the lens.

The image formed by the object glass can be seen by the naked eye, but it is very small, and a distant object cannot be distinguished because its image is so minute. The eyepiece is a microscope which magnifies the image and causes it to appear larger and therefore nearer than the object itself as seen with the naked eye. It enables the observer to distinguish a signal staff, or target, or other object, which to the unaided eye would be invisible. An erecting eyepiece is a combination of lenses, usually four in number, which reverses the image and causes it to appear erect to the observer. An inverting eyepiece is a combination of lenses, usually two in number, which does not reverse the image, and the latter is therefore seen inverted.

The particular line in the telescope which is to be brought into coincidence with the line of sight to the required point is defined by the cross wires. These are two fibers of cobweb or of the finest platinum wire fixed at right angles to each other across a ring and placed in the telescope within focusing range of the eyepiece and object glass, both of which are mounted in tubular slides operated by rack and pinion movements. The eyepiece is first focused to give a clear and distinct view of the cross wires. Then the object glass slide is moved until the image of the object observed falls in the plane of the cross wires, when it will also be in the focus of the eyepiece, and will be distinctly seen. If, when the eye is moved slightly from side to side, the wires appear to move on the image, the focusing of the object glass is not correct, and it must be readjusted until no such movement is apparent.

Now the telescope may be so moved and pointed that the intersection of the cross wires will fall exactly on the image of the point observed, and when this is done it is known that the intersection of the cross wires, the center of the object glass, and the point observed, are in the same straight line, viz., in the line of sight to, or ray of light from the point observed. In other words the line of collimation of the telescope has been made to coincide with the line of sight to the point observed.

The line of collimation of the telescope is the straight line through the intersection of the cross wires and the center of the object glass. It coincides with the undeviated ray that falls on the intersection of the cross wires.

When the intersection of the cross wires is made to coincide with the image of a point, the telescope is said to be pointed at or directed toward that point.

The tube of the telescope holds the parts in their proper relative positions, protects them from injury, and shuts out unnecessary light.

The Horizontal Axis.—The telescope is mounted on trunnions the axis of which intersects the line of collimation perpendicularly, and which rest horizontally in trunnion beds on standards high enough to permit complete revolution of the telescope in the vertical plane. The standards are fastened to a horizontal plate.

The Vertical Axis.—The circular plate that carries the telescope has a central stem or spindle whose axis is vertical and, if prolonged, passes through the intersection of the horizontal axis and the line of collimation. The spindle fits and turns in a socket in an outer hollow spindle and thus permits complete revolution of the telescope, in a horizontal plane.

The Circular Arcs.—A graduated circle or semicircle is attached centrally and perpendicularly to one of the trunnions of the telescope, and turns with the telescope in its vertical revolution. Its index and vernier are fastened to the standard and remain fixed. By means of this circular arc and its vernier, vertical angles may be read.

The outer hollow spindle is expanded at the top into a saucer-like plate, on the upper rim of which is the horizontal circular arc with its center in the vertical axis. This lower plate marked with the graduated circle remains fixed, while the upper plate carrying the telescope revolves on the vertical axis and thus turns off horizontal angles that may be measured on the arc. The index and the vernier are fastened to the upper movable plate. Most transits have two verniers placed diametrically opposite to each other. By taking the mean of the readings of both verniers, any error due to eccentricity of the circle is eliminated.

The outer spindle, which carries the lower plate, fits vertically in a socket, in which it may be turned to set or "orient" the lower plate and circle in any desired position. When set or oriented it must be fastened or clamped in the socket to prevent further motion.

This socket supports and centers all the working parts of the instrument and must itself be firmly supported, with its axis vertical. The supports provided are leveling screws arranged as follows: The socket is fastened in a horizontal frame or flange which is pierced by three or four vertical screws having milled heads. These screws rest on a fixed base plate and form adjustable supports for the instrument, by means of which the axis of the instrument may be made vertical. The verticality of this axis is determined by two spirit levels fastened horizontally and at right angles to each other above the plates. One level would be sufficient for this purpose, but two are generally used in order that the instrument may be leveled without having to turn it on its vertical axis.

The leveling screws support but do not fasten the socket of the instrument on its base plate. For this latter purpose there is provided a ball and socket joint when four leveling screws are used, or a spiral spring when three leveling screws are used.

The base plate is often made double, so that one part will slide upon the other and permit slight lateral motion of the instrument for exact adjustment over the given point. This arrangement is called a *shifting center.*

The base plate is screwed on to the top of the *tripod,* which is a stand with three legs hinged at the top and arranged to support the instrument at a convenient height.

When it is desired to set up the instrument on a wall or stump or on a high framed station, use is made of a *trivet*, which is a brass crowfoot with three points of support.

Clamps and Tangent Screws.—The motions of revolution of the instrument are controlled by clamps and tangent screws. The clamp is a ring or a segment of a ring which turns freely on one of the two parts to be connected, but which can be fastened to that part by a clamp screw having a milled head. The ring is extended into an arm that is permanently fastened to the other part. In this manner a ring on the trunnion of the telescope is fastened by its arm to the standards, a ring on the spindle of the lower plate is fastened by its arm to the upper plate, and a ring on the same spindle of the lower plate is fastened to the socket. Any motion of the instrument may therefore be clamped or released at will by tightening or loosening the corresponding screw.

In each of the arms that join the parts, there is inserted a tangent screw connection which gives a very slow motion to the part affected, and permits accurate pointing of the telescope at the point observed or accurate setting of the verniers at any desired reading.

The Plumb Line.—A known point on or near the ground is usually marked by a stake with a tack driven in its top. To read horizontal angles at this point the transit must be set up with its center exactly over the point. A plumb line is hung from the center of the spindle and the instrument is so placed that the point of the plumb bob is at the center of the tack or mark. The vertical axis of the instrument will then pass through the given point.

ATTACHMENTS OF THE TRANSIT.

The following named parts are not essential parts of the transit proper, but are attachments that serve some useful or convenient purpose:

The Telescope Level is a level tube placed beneath the telescope and attached to it. The axis of the level tube should be parallel to the line of collimation of the telescope and should be in the same vertical plane. With this attachment the transit may be used as a surveyor's level.

THE STADIA.

The *stadia wires* are two additional horizontal wires that are placed one above and the other below, at equal distances from the horizontal cross wire of the telescope. They are used in connection with the stadia rod to measure the distance from the center of the instrument to any unknown point at which the stadia rod may be placed.

The principle of the stadia in its simplest form is as follows:

With the eye at the peep-hole E (Plate 1, Fig. 1), sighting past the fixed wires W and W′, there will be intercepted on a rod at A, the space S S′, and on a rod at B, the space T T′. Since the triangles E S S′ and E T T′ are similar, the distances E A and E B are proportional to the intercepts S S′ and T T′. If the intercept S S′ is one foot at a distance E A of 100 ft., then any other intercept, as 3.53 ft. on the rod would indicate a distance of 353 ft. from E to the rod.

When the stadia wires are placed in the telescope the principle of similar triangles still applies, but the intercepts on the rod are proportional to distances measured from a point in front of the lens and not from the eye. This may be explained as follows:

A law of lenses is that the reciprocal of the principal focal length is equal to the sum of the reciprocals of the conjugate focal lengths, or

$$\frac{1}{f} = \frac{1}{f_1} + \frac{1}{f_2} \quad \text{..................(1)}$$

in which f is the principal focal length and f_1 and f_2 are the conjugate focal lengths, that is, f_1 is the distance from lens to focus for rays of light coming from a point at a distance f_2 from the lens, and f is the distance from lens to focus for parallel rays or rays from a point at an infinite distance.

Let I (Plate 1, Fig. 2) be the center of the transit, A′ and B′ be the positions of the stadia wires, L be the position of the lens, and A and B be the points whose images fall at A′ and B′ respectively, and let

s = A B = intercept on rod,
i = A′ B′ = image of intercept,
$\left.\begin{array}{l} f_1 = F' L \\ f_2 = F\ L \end{array}\right\}$ conjugate focal lengths,
c = I L = distance from center of instrument to center of lens,
f = L D = principal focal length,
d = I F = required distance from center of instrument to rod.

From similar triangles,

$$f_1 : f_2 :: i : s, \therefore \frac{1}{f_1} = \frac{s}{i f_2} \quad \text{.............................(2)}$$

From equation (1), $$\frac{1}{f_1} = \frac{1}{f} - \frac{1}{f_2} \quad \text{..........(3)}$$

Eliminating $\frac{1}{f_1}$ and solving for f_2,

$$f_2 = f + \frac{f}{i} s \quad \text{..........(4)}$$

which shows that the distance f_2 from the *lens* to the rod is equal to the constant distance f, plus s multiplied by the constant ratio $\frac{f}{i}$. Also, that if the distance f be laid off from L to D, the remaining distance D F from D to the rod is proportional to the intercept s.

This is illustrated in Plate I a, Fig. 2, where three different positions of the rod A B are shown with the corresponding positions A′ B′ of the stadia wires when focused on the rod. The triangles behind the lens, having equal bases A′ B′ and different altitudes are dissimilar. Therefore the corresponding triangles in front of the lens having the rod intercepts A B for their bases and their apices at L are also dissimilar, and distances from the lens to the rod are not proportional to the rod intercepts. But straight lines drawn through the extremities of the rod intercepts will meet at the point D, and distances from this point are proportional to the corresponding rod intercepts. The point D is at a distance from the center of the lens equal to f, the principal focal length of the lens.

In most instruments, it is the lens that is moved in focusing the telescope for different distances, but the figure is simplified by supposing that the stadia wires are moved, as is the case in some telescopes. The motion is so slight that it may be disregarded except in explaining the theory.

To get the total distance d from the center of the instrument I to the rod, the small distance c from I to L (Plate I, Fig. 2) must be added, and there results—

$$d = c + f + \frac{f}{i} s \text{..................} (5)$$

f is found by measuring the distance from the lens to the cross wires when focused on a star or on a distant horizon. c is found by measuring the distance from the horizontal axis to the lens when focused on an object whose distance is a mean of the expected readings. The slight variation in c due to the movement of the lens in focusing for different distances is neglected. f and i cannot be measured with sufficient accuracy to determine the ratio $\frac{f}{i}$.

Equation (4) may be written in the form

$$\frac{f}{i} = \frac{f_2 - f}{s} \text{..........} (6)$$

and the ratio $\frac{f}{i}$ may be found by measuring s for some particular measured value of f_2.

On level ground, measure from the plumb bob of the transit the distance $(c + f)$ and from the apex D, so determined, lay off in the same direction a base

$(f_2 - f)$ of say 400 feet. At this distance a rod will be held vertically and the observer will signal the rod-man to set a target or a pencil point, first at the point cut by the lower wire and then at the point cut by the upper wire. The intercept s between these two points is carefully measured in feet; then the base $(f_2 - f)$ divided by the intercept (s) gives $\frac{f_2 - f}{s}$ which is equal to $\frac{f}{i}$. If with a base D L of 400 feet the intercept is 3.965 feet, the ratio $\frac{f}{i}$ will be $\frac{400}{3.965} = 100.88$.

Now if the rod be held at any other point, the new intercept s may be measured on the rod and the distance from the apex D to the rod will be $100.88 \times s$. The total distance from the center of the instrument would be

$$d = (c + f) + (100.88 \times s).$$

$(c + f)$ is a known constant and s, the intercept, is seen on a graduated rod between the stadia wires of the telescope. The rod is so plainly marked that the observer can read its divisions through the telescope. Multiply s by the determined ratio and add $(c + f)$. The result is the required distance from the center of the instrument to the rod.

If the rod is graduated in feet this multiplication must be performed for every distance read, but if a scale be constructed on the rod with a unit equal to the intercept at 100 feet from the apex D, and with decimal subdivisions then the rod readings increased by $(c \times f)$ will give distance directly, and the labor of multiplying or of referring to a table is avoided.

To mark a stadia rod in this manner, suppose as before that the intercept on a rod held at a distance of 400 feet from the apex D is 3.965 feet. One-fourth of this or 11.89 inches would be the intercept at 100 feet, and is the unit to be laid off on the rod to represent 100 feet. The rod should be a light smooth board about ½ inch thick, 4½ inches wide, and 12 to 15 feet long, painted white. The rod may be stiffened by attaching a longitudinal rib on the middle line of the back. On the face of the rod mark the center point and lay off the unit (11.89 inches in the case now considered) successively each way from the center on the middle line toward the ends. Subdivide each unit into twenty equal parts, each representing 5 feet. Draw cross lines with a try-square and sharp pencil at each point of division and then paint red or black marks or designs, making strong characters that can be plainly seen through the telescope at a distance.

The best marking for a stadia rod is that shown in Plate 1, Fig. 3. The spaces representing 100 feet are distinct black masses separated by white, and can be readily counted even at a distance. The 50-ft. points are plainly marked by the large reëntrant angles and the 25 and 75-ft. points by the double salients. The 10-ft. points are the small reëntrants and the 5-ft. points are the small salients. The five-foot intervals may be subdivided by eye and read by estimation to the nearest foot or half foot.

To read the rod set one wire at an exact 100-ft. division and count the whole hundreds up to the one that is cut by the other wire and add the tens and units to the exact point cut by the second wire. Thus, if the lower wire is at A and the upper at B the reading of the rod is 231 ft. and if $(c + f)$ is 0.8 ft. the total distance to the rod is 231.8 ft.

If a rod is to be marked to read yards, it is convenient to avoid too large a figure and to take as the unit the intercept that corresponds to a distance of 40 yards. Subdivide and mark in the same manner as is shown for the 100-ft. division. The smallest divisions will then read to 2 yards, and, by estimation, to the nearest foot.

For meters, proceed in the same manner as for yards.

If the rod be held on a point that is higher or lower than the instrument the line of sight will be inclined, and to get a correct reading the rod should be held perpendicular to the line of sight. To do this is generally impracticable or at least inconvenient. Moreover, it is not the actual distance along the line of sight that is required, but rather the horizontal distance and the difference of elevation between the two points. Therefore the rod is held vertically and its reading and the angle of elevation or depression to the required point are taken. A reduction is then made to get the horizontal distance and difference of elevation between the two points. The formulæ for this reduction are deduced as follows: (Plate I, Fig. 4.)

Let K be the known point at which the transit is set up.

P be the required point at which the stadia rod is placed vertically.

I be the center of the transit, and F be a point on the rod such that the height F P is equal to I K.

Knowing the rod reading, the vertical angle A, and the angle subtended by the stadia wires 2 B, required the horizontal distance K L or I H, and the difference of elevation, L P or H F, between the points K and P.

Let $b = $ K L or I H, the horizontal distance.

$a = $ L P or H F, the difference of elevation.

$s = s' + s''$, the distance read on the vertical rod.

$2r = TT' =$ the rod reading or true distance if the rod were perpendicular to the line of sight at F. This rod reading would be the distance D F or $d - (c+f)$. Then

$$b = (c + f + 2r)\cos A.$$

$$\text{and } a = (c + f + 2r)\sin A.$$

To find $2r$; in the triangle F T R

$$s' : r :: \sin(90^\circ + B) : \sin(180^\circ - A - 90^\circ - B):$$

and in the triangle FT′R′

$$s'' : r :: \sin(90^\circ - B) : \sin(180^\circ - A - 90^\circ + B):$$

Whence

$$s' = r\frac{\cos B}{\cos(A+B)} \ldots\ldots(1), \text{ and } s'' = r\frac{\cos B}{\cos(A-B)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

$$s = s' + s'' = r\frac{\cos B}{\cos(A+B)} + r\frac{\cos B}{\cos(A-B)} \ldots\ldots\ldots\ldots\ldots(3)—$$

$$2r = s(\cos A - \sin A \tan A \tan^2 B) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

$$b = (c + f)\cos A + s(\cos^2 A - \sin^2 A \tan^2 B) \ldots\ldots\ldots\ldots\ldots(5)$$

$$a = (c + f)\sin A + s(\sin A \cos A - \sin^2 A \tan A \tan^2 B) \ldots\ldots(6)$$

B is a small angle equal to one-half the angle subtended by the stadia wires. $\tan^2 B$ is therefore a very small quantity and the second term in the parenthesis in each of equations (5) and (6) may be neglected without appreciable error. In most transits the intercept of the stadia wires at 100 ft. is about 1 ft., hence the tan B $= \frac{1}{200}$ and $\tan^2 B = \frac{1}{40000}$. Neglecting this small quantity the formulæ become

$$b = (c + f)\cos A + s\cos^2 A \ldots\ldots\ldots\ldots(7)$$

$$a = (c + f)\sin A + s\sin A\cos A \ldots\ldots\ldots(8)$$

The values of *b* and *a* derived from these formulæ are tabulated and the tables are published in works on surveying or in separate volumes. One part of the table gives the values of the first terms for (c + f) equal to the usual values from 0.75 ft. to 1.25 ft., and for angles A from zero to 20 or 30 degrees Another part of the table gives the values of the second terms for s = 100 and for angles A from zero to 20 or 30 degrees. For other values of s multiply the tabular number corresponding to the angle A by $\frac{s}{100}$.

When great accuracy is not required a method of marking the rod that is only approximately correct is adopted. The small distance, (c + f), is neglected. The base is measured from the vertical axis (or plumb line) of the transit and at the farther end of the base a rod is held on which is marked the intercept of the wires. This intercept divided by the number of hundreds of feet in the base gives the unit to be used in marking the rod. A rod so marked will give a correct reading only for the distance equal to the length of the base. At lesser distances the readings will be too small and at greater distances the readings will be too large.

For, let A (Plate II, Fig. 1) be the center of the transit and let A B be the measured base. The intercept at B is measured and the rod is graduated as already explained. If this rod could be seen through the telescope when held at C, the true apex, the intercept would be zero and the distance would be called zero, whereas it is actually (c+f). The reading is therefore too small by (c+f). If B E is made equal to B C, the intercept at E would be G H, which corresponds to the distance A E′, too great by (c+f). The error that is made at any point in using a rod so graduated is, therefore,

$$e = (c+f)\frac{b-s}{b},$$ in which e = the error, (c+f)

is the constant of the instrument, b is the measured base and s is the rod reading or observed distance.

Notwithstanding the error that is introduced, this method of graduating stadia rods is often used in order to avoid the labor of adding the (c+f) constant or its reduction to every reading; but since the stadia will measure distances with an accuracy of about 1 in 1000, the introduction of an error which may be cumulative should be avoided.

The following method of graduating stadia rods is correct for horizontal readings and only slightly in error for readings reduced to the horizontal or vertical by the usual tables or diagrams.

The formula $d=c+f+\frac{f}{i}s$ may be written in the following form:

$$d = \frac{f}{i}\left[(c+f)\frac{i}{f} + s\right],$$

that is, the distance d is proportional to the intercept, s, increased by the constant $(c+f)\frac{i}{f}$. Having graduated the rod accurately by the first method lay off this constant $(c+f)\frac{i}{f}$ on either side of each even 100-ft. division and fill in the space so marked with black at the sides of the rod as shown in Plate 1, Fig. 3.

In reading the rod, set the lower wire on the upper line of this black mark, but read from the adjacent 100-ft. division. The effect of this is to add to every intercept a rod distance of $(c+f)\frac{i}{f}$ which corresponds to a ground distance of (c+f). The rod reading, therefore, includes the correction (c+f). It is just as easy to set the initial wire at the (c+f) mark as it is to set it at the exact 100 ft. mark and no further correction for (c+f) is required.

In reducing inclined readings of this rod to the horizontal by the formula

$$b = (c + f) \cos A + s \cos^2 A$$

the first term, (c+f) cos A, disappears because the (c+f) is included in the rod reading, s, and here a small error is introduced by multiplying the (c+f) part of s by $\cos^2 A$ whereas it should be multiplied by cos A only. But (c+f) seldom exceeds 1 ft. and cos A does not differ much from unity because the angle A seldom exceeds 15 degrees. For this angle cos A = 0.966 and $\cos^2 A = 0.933$. Hence the error, for c+f = 1 ft. and A = 15°, is only 0.033 ft. which is negligible in stadia work.

In applying the formula

$$a = (c + f) \sin A + s \sin A \cos A$$

to obtain differences of elevation the first term disappears because (c+f) is included in s, and an error arises from multiplying the (c+f) part of s by sin A cos A, whereas it should be multiplied by sin A only. But the error is very small. If A = 15 degrees, sin A = 0.259 and sin A cos A = 0.250; hence for (c+f) = 1 ft. the error is only 0.009 ft. Generally the vertical angle is less than 15 degrees and the errors are correspondingly smaller.

With *this rod*, therefore, horizontal distances are read correctly and inclined readings are reduced to the horizontal and vertical, without appreciable error, by taking the values from the usual tables or diagrams without reference to the $(c+f)$ corrections. The $(c+f)$ part of the table may in this case be totally disregarded.

THE COMPASS.

The transit usually has a compass mounted centrally on the upper plate. It is similar to the surveyor's compass and will be described under that head.

THE VERNIER.

A *measuring scale* is a line divided by cross marks into equal parts, each part being a unit of the magnitude to be measured. In surveying, such scales are used for measuring distances and angles.

For distances the scale is marked on a chain or tape or rod, and the parts or divisions are usually feet subdivided into tenths and hundredths, or meters subdivided into centimeters.

For angles the scale is a circle divided into 360 equal parts, each part being the measure of one degree. Each part may be subdivided into two, or three, or four, or six smaller parts, each measuring 30, or 20, or 15, or 10 minutes of arc, respectively.

The divisions of a scale are usually numbered from an assumed origin or zero mark to facilitate counting the number of divisions included in the distance or angle to be measured.

The index of a scale is a mark which indicates the point at which the scale is to be read. Thus, if a tape be stretched with its zero at one mark on the ground and its edge passing through another mark on the ground, this second mark is the index showing the exact point at which the scale is to be read. In measuring angles, the index is a mark or arrow head which moves along the circular scale for different pointings, and for any pointing shows the exact point at which the scale is to be read.

When the index happens to fall exactly at one of the marks on the scale, the number of that mark is the reading of the scale. When the index falls between two marks on the scale a fractional part of a division is to be added to the number at the mark which the index has passed. This fractional part may be estimated by eye with considerable accuracy.

A *vernier* is a device for measuring the fractional part of a division of a scale from the index back to the preceding mark of the scale.

Fig. 1, Plate I a, is a portion of a linear scale showing the 4.0, 4.1, 4.2, 4.3, and 4.4 ft. marks, with each tenth of a foot divided into ten equal parts, which are therefore hundredths of a foot. The zero or origin of the scale is to the left. If the index, or point at which the scale is to be read, be at A, it is seen that it is exactly at one of the division marks of the scale, and the reading is 3.980 feet. If the index be at the point B, the scale reads 4.12 ft. and a fraction over. This fraction may be estimated by eye to be about seven tenths of a division beyond the 4.12 mark, giving a reading of 4.127 ft., but to measure this fractional part of a division a vernier is used. Return to the point A and observe that an auxiliary scale is constructed to the right of the index, that its length covers nine divisions of the main scale, and that it is divided into ten equal parts. Then each of these parts is one tenth of nine divisions or nine tenths of one division, or each part is smaller than a division of the main scale by one tenth of a division. If the index and vernier be moved to the right one tenth of a division, the first mark of the vernier will coincide with a mark on the scale. If the index be moved two tenths of a division, the second mark of the vernier will coincide with a mark on the scale, and so on. With index at B, it is the seventh mark of the vernier that coincides with a mark of the scale and this shows that the index has passed seven tenths of a division beyond the 4.12 mark, giving a reading of 4.127 ft. This vernier reads in the same direction as the scale and is called a direct vernier.

At C is shown another form of vernier constructed to the left of the index. Observe that its length covers exactly eleven divisions of the scale, and that it is divided into ten equal parts, each part being equal therefore to eleven tenths of a division of the scale or one tenth longer than a division. The index at C is a little beyond the 4.40 ft. mark, bringing the second mark on the vernier into coincidence with a mark on the scale. This shows that the index has passed two tenths of a division beyond the 4.40 mark, giving a reading of 4.402 ft. This vernier reads to the left or in a direction opposite to the readings of the scale, and is called a retrograde vernier.

In general, if a vernier have n divisions, its *least reading* is $\frac{1}{n}$ of a division of the main scale. If it cover $n - 1$ divisions of the scale, it is direct; if it cover $n + 1$ divisions of the scale, it is retrograde. Or if the vernier divisions be smaller than the scale divisions, it is direct; if larger, it is retrograde.

It is convenient to have the zero of the vernier at the index, but this is not necessary provided always that the zero mark of the vernier and the index have simultaneous coincidence with marks on the scale.

Any mark on the vernier may be taken as the zero mark. Sometimes the middle mark is so taken, and the vernier reads from the middle to one end, and continues from the opposite end back to the middle.

On many transits the limb is graduated or numbered in both directions, and the vernier must also be numbered in both directions, one set of numbers to be used for reading the limb in one direction and the other set for the opposite direction; or else two separate verniers are provided.

If a limb of a transit be divided into degrees and half degrees its smallest reading is 30 minutes. A vernier to read minutes would have 30 divisions. If it covered 29 divisions of the limb it would be direct; if it covered 31 divisions it would be retrograde. Direct verniers are commonly used.

On a circular arc the degrees are subdivided into three equal parts, each measuring 20 minutes. Its vernier has 40 divisions and covers 39 divisions of the arc. What is the least reading of the vernier? Is it direct or retrograde?

A circular arc is divided into degrees. A vernier is to be constructed having a least reading of five minutes. How many divisions will the vernier have and how many divisions of the arc will it cover if direct? How many if retrograde?

A linear scale is marked off in meters and centimeters. Its vernier has ten divisions. What is the least reading? If direct, how many centimeter divisions does the vernier cover?

PLATE I.

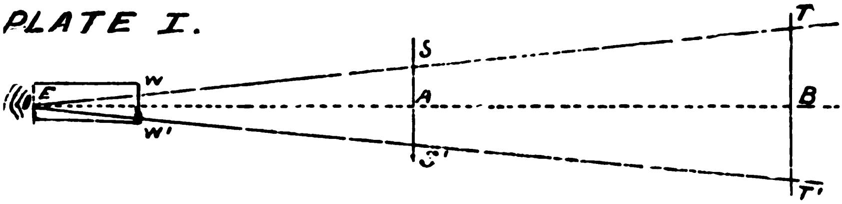


Fig. 1

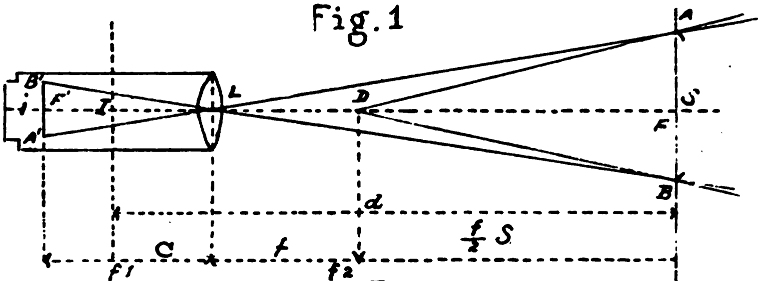


Fig. 2

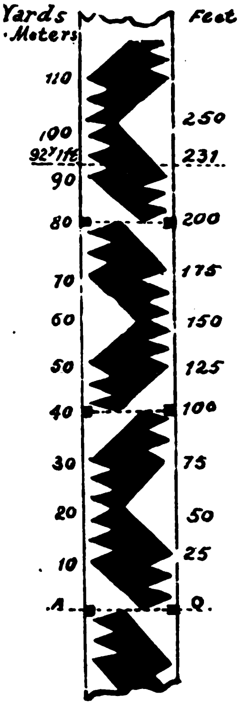


Fig 3

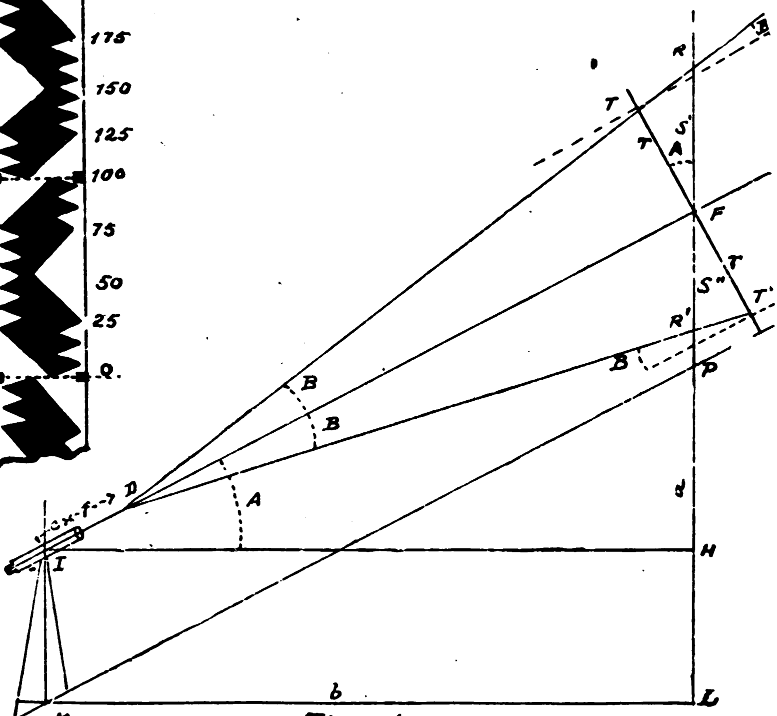
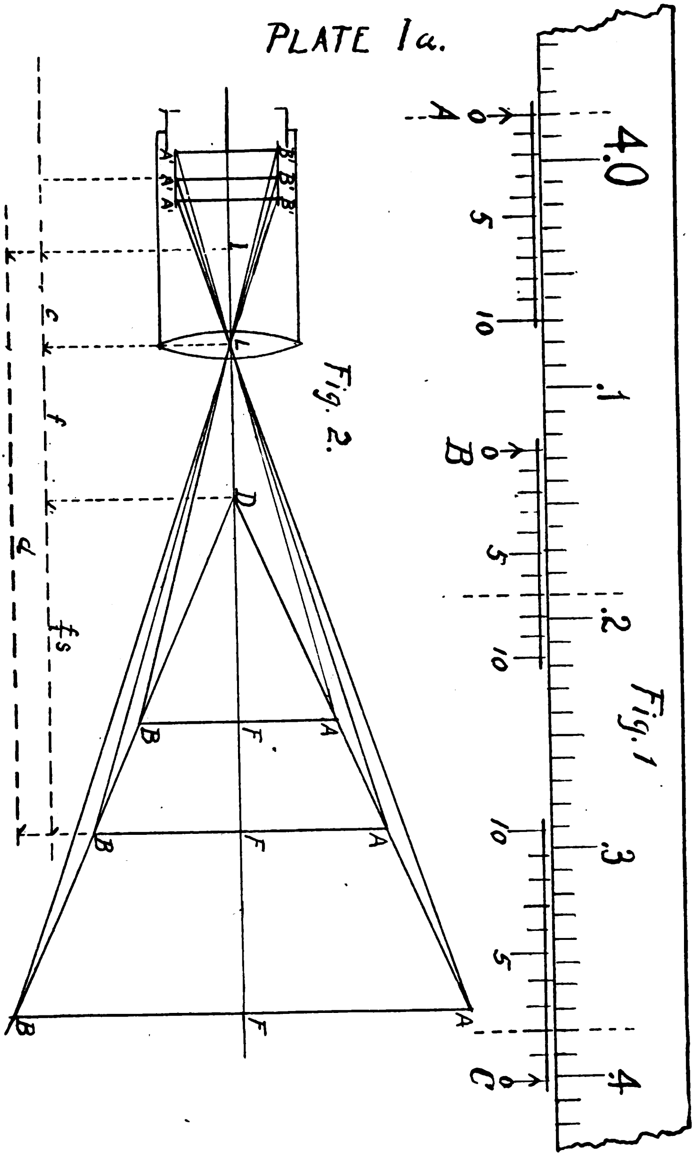


Fig. 4

PLATE Ia.



CHAPTER IV.

ADJUSTMENTS OF THE TRANSIT.

There are certain parts of the transit, as well as of other instruments, that the makers are unable to fix permanently and rigidly in their proper places with certainty and exactness. Such parts are made movable by means of opposing screws or otherwise, and their adjustment consists in bringing the movable parts into their proper positions.

The parts of the transit that are adjustable are the plate levels, the cross wires, the horizontal axis, the telescope level, and in some instruments the vernier of the vertical circle, the object glass slide, and the eyepiece.

The adjustments are:—

1. To make the axes of the plate levels perpendicular to the vertical axis.
2. To cause the line of collimation to revolve in a plane.
3. To cause the line of collimation to revolve in a vertical plane.
4. To make parallel the axis of the telescope level and the line of collimation.
5. To make the vernier of the vertical circle read "zero" when the line of collimation is horizontal.
6. To center the object glass slide.
7. To center the eyepiece.

The adjustments of the attached compass are the same as those of the surveyor's compass and will be considered under that head.

THE LEVEL TUBE.

A level tube is a tube of glass so bent and ground that a longitudinal vertical section of the upper interior surface is an arc of a circle convex upward. This tube is nearly filled with ether and is sealed. The unfilled space forms a bubble of ether vapor which always seeks the highest part of the tube. A plane tangent to the upper surface of the bubble at its middle point is therefore a horizontal plane.

A line tangent to the longitudinal section of the upper inside surface of the tube at its middle point is called the axis of the tube. When the middle of the bubble is at this middle point of the tube this axis of the tube is horizontal.

If a level tube be revolved about a vertical axis, every point of the tube will revolve in a horizontal plane and that point of the tube that was highest will remain highest throughout the revolution. Therefore the bubble will not move from its original position in the tube.

Conversely, if a level tube be revolved about an axis and the bubble does not move from its original position, the axis of revolution is vertical. If the bubble is at the center of the tube and remains there during the revolution, the axis of the tube is horizontal and is perpendicular to the vertical axis of revolution.

If a level tube be revolved 180° about an axis not vertical or be reversed 180° on fixed points of support the motion of the bubble in the tube, measured in degrees of arc, will be twice the angle made by the axis with the vertical or by the line supports with the horizontal respectively. For: Let A C (Plate II, Fig. 2) be the level tube, P O be the axis of revolution, and B O be a vertical line through the center O of the arc of the tube. The bubble will be at the highest point B. After revolution of 180° about P O, the point of the tube that was at B will be found at D, and the point that was at D will be found at B, B D being perpendicular to P O, but the bubble will remain at the highest point B and will therefore have traveled in the tube over the arc D B equal to twice the arc P B that measures the inclination of the axis to the vertical. To make the axis vertical, it and the tube must be turned in their own plane till the bubble is at P the middle point of the traveled arc. Having made the axis vertical, the tube may be moved independently till the bubble is at its middle point, and the axis of the tube will then be perpendicular to the axis of revolution and horizontal.

Reversing the level tube 180° on two fixed points of support is equivalent to revolving it 180° about an axis that is perpendicular to the line joining the points of support, and the same demonstration holds good.

The foregoing principles are applied in all adjustments of level tubes, and in all cases where a level tube is used to determine a horizontal or vertical line.

First Adjustment of the transit: To make the axes of the plate levels perpendicular to the axis of the instrument: First, make the axis of the instrument vertical and then adjust the plate levels so that their bubbles shall be at the middle points of the tubes.

For the first part of this adjustment it is best to use the telescope level because it is more sensitive and accurate than the plate levels. Level the instrument approximately, and turn the head to bring the telescope level into the vertical plane containing two opposite leveling screws. Bring the bubble to the middle of its run and note its exact position. Reverse the instrument 180° on its vertical axis, and when the bubble has come to rest note its new position. With the leveling screws bring the bubble back over one-half of its displacement. This will bring the vertical axis into a vertical plane that is at right angles to the axis of the bubble. Now turn the head of instrument 90° to bring the level into the plane containing the other pair of leveling screws, and repeat the operations described above. This will bring the axis of the instrument into another vertical plane, and since it lies in two vertical planes it must be their line of intersection and be vertical. The foregoing operations should be repeated in both positions as a check.

The axis of the instrument being now vertical, adjust the plate levels by means of the capstan headed screws, and bring their bubbles to the middle mark of the tube. The axes of the plate levels will then be perpendicular to the vertical axis, and when the instrument is again set up, and leveled by bringing these bubbles to the middle of their tubes, the axis of the instrument will be vertical.

Second Adjustment: To cause the line of collimation to revolve about the horizontal axis in a plane. It will do this only when it is perpendicular to the horizontal axis. If it is not perpendicular thereto it will in its revolution generate a cone and not a plane.

Select a place where the ground is clear and approximately level in opposite directions from the point to be occupied by the transit. Set up and level the transit. (1) Direct the cross wires upon a well defined point in any direction which may for reference be called east. It will be best to drive a stake at a distance of about 200 feet with a half driven nail in its top. Bisect the nail with the cross wires, clamp both plates, and (2) plunge the telescope to the west. About 200 feet from the transit in this direction, an assistant will drive a second stake, being signaled by the observer to place it in the line of sight through the cross wires. The assistant will then hold a pencil point on top of the stake and move it as indicated by the observer till it is bisected by the cross wires. The point so determined is marked by a dot on top of the stake. (3) Reverse the transit on the vertical axis and point again at the eastern point, bisecting the nail accurately. Clamp the plates and (4) plunge the telescope again to the west. Signal the assistant to set the pencil again at the point intersected by the cross wires and mark the point on the top of the stake. If this point does not fall on the stake, another stake must be driven and the point marked on it, but usually the second pencil point will fall near the first, and if a broad stake has been used both points will fall on its top. Draw the line connecting the two points and mark a point at a distance from the second pencil point equal to one-fourth the length of the line. This last point will lie on a line that intersects the vertical axis of the transit and is perpendicular to the horizontal axis of the telescope in its last position. If the cross wires be moved by their adjusting screws till their intersection falls upon this last point the line of collimation will be made perpendicular to the horizontal axis.

For, let T (Plate II, Fig. 4) be the position of the transit, and (1) let T E be the direction of the line of collimation when it first bisects the east point E, and let P T P′ be the perpendicular to the horizontal axis. (2) When the telescope is plunged the line of collimation will fall on T W′ the angle W′T P′ being equal to PT E. (3) Now the transit is reversed through the angle W′T E and the line of collimation again bisects E. The perpendicular to the horizontal axis will revolve through the same angle and fall in P″ T P‴, the angle P″ T E = W′ T P″ = P T E. (4) Plunging the telescope again to the west lays the line of collimation on T W_2 with the angle W_2 T P‴ = P″T E. Prolong the line E T to W, and the angles P′T W and P‴T W will be equal respectively to P T E and P″T E, therefore equal to each other and to P′ T W′, and to P‴ T W_2, which last angle is the angle of deviation of the line of collimation from the perpendicular to the horizontal axis in its last position. The points W′ and W_2 have been marked by pencil dots. Mark P‴ at a distance from W_2, equal to one fourth of the line W_2W′. Then adjust the cross wires so that their intersection shall fall upon P‴ and since T P‴ is perpendicular to the horizontal axis the line of collimation will also be perpendicular to that axis.

To test the adjustment mark the middle point W of the line W′ W_2. Turn the telescope on E and bisect accurately, then plunge the telescope, and if the cross wires bisect W the adjustment is correct. If not repeat the operations above described till the adjustment is completed.

Third Adjustment: To cause the line of collimation to revolve on its horizontal axis in a vertical plane. It will do this only when the horizontal axis is truly horizontal.

First Method: Set up the transit about 50 or 60 feet from the wall of a building and point the telescope approximately in a plane perpendicular to the wall. Direct the cross wires accurately upon a well-defined point near the top of the building and clamp both plates. Plunge the telescope downward to a point near the ground, and have an assistant mark with pencil or chalk the point on which the cross wires fall. Reverse and plunge the telescope and direct the cross wires again at the upper point. Clamp the plates and again plunge the telescope down to a point near the ground at the same height as the other point. The assistant will mark this second point. If it falls upon the first marked point the adjustment is correct. If not, the horizontal axis is shown to be inclined, and the higher end is toward the side corresponding to the second marked point. Mark the point that is midway between the two marks. This middle mark and the upper point will lie in the vertical plane through the center of the transit. The line of collimation, when it is plunged downward from the upper point should pierce this middle mark. With the adjusting screws of the standard, lower the high end or raise the low end of the horizontal axis. Sight again at the upper point, clamp the plates and plunge downward to the lower points. Note how much of the correction has been made, and adjust till by estimation the remaining part of the correction has been effected. Repeat the test and readjustment until the line of collimation can be made to pierce both the upper point and the lower middle mark, in its revolution on the horizontal axis.

To explain this adjustment it is only necessary to show that the lower middle mark is in the vertical plane through the center of the transit and the upper point. Let C (Plate II, Fig. 3) be the intersection of the vertical and horizontal axis of the transit, shown in projection on the plane of the wall, and assume that the right hand standard is the higher, giving A B as the projection of the horizontal axis. Let P be the upper point on the building, lying in the vertical plane through C perpendicular to the wall. When the telescope is pointed at P and then plunged downward, the line of collimation will trace on the wall the line P M′, perpendicular to the projection A B of the horizontal axis. When the telescope is reversed and plunged, and pointed again at P, the higher standard will be on the left and the horizontal axis will be projected in A′ B′ making the angle P C B′ equal to P C B. When the telescope is again plunged downward, its line of collimation will trace on the wall the line P M″ perpendicular to the projection A′ B′ of the horizontal axis. Since the angles B C P and B′ C P are equal, their complements M′ P C and M″P C respectively are equal, and if M′ and M″ lie in the same horizontal, a point M, midway between them, will lie in the vertical plane through P and C.

The foregoing demonstration is not mathematically exact, but the errors balance and the result is correct.

Second Method: Sight at any well defined high point, as the gable of a roof or the corner of a chimney, and clamp the plates. Place a bucket or pan of water on the ground in front of the transit in such a position that the observer can see in the water a reflected image of the selected point. Plunge the telescope downward and sight at this image. If the cross wires bisect it the adjustment is correct. If not, make such adjustment of the standard as will cause the cross wires to bisect both the selected point and its image when turned on the horizontal axis from one to the other.

The lines of sight from the instrument to the selected point and to its image in the water lie in the same vertical plane, and when the line of collimation is made to revolve in that plane the axis of revolution is horizontal and the adjustment is correct.

Fourth Adjustment: To make parallel the axis of the telescope level and the line of collimation:

First, make the line of collimation horizontal, then adjust the level so that its axis shall be horizontal (bubble in center) and the two lines will be parallel.

First Method: An understanding of this method depends upon a knowledge of the principles of "leveling," and beginners should revert to this method after they have studied the "level" and its use. Drive two stakes about 100 feet apart to serve as points of support for a vertical measuring rod. The ground between the stakes should be approximately level and free from obstructions. Call one stake the Bench Mark (B. M.) (Plate III, Fig. 1.), and the other the Turning Point (T. P.). Set up the transit on the line joining the two stakes and close to the B. M. stake so that the eye end of the telescope shall be about ½ inch from a measuring rod held vertically on the stake. Level the transit and bring the bubble of the telescope level to the middle of the tube. With the eye at the object glass, sight backward through the telescope and mark with a pencil dot or a target the point on the rod seen at the center of the small field of view. The height of this dot as measured on the rod above the B. M. is called a Back-sight (B. S.). Denote this height by "a," and enter it in the record in the column "B. S." opposite B. M. in column "Station."

Sta.	B. S. +	H. I.	F. S. —	Elev.
B M	a	a		o
T P			b	a — b
T P	a′	a — b + a′		
B M			b′	a — b + a′ — b′

If the elevation of the B. M. be assumed as zero, "a" will be the "height of instrument" (H. I.) or of the line of collimation above the B. M., and is entered in column H. I.

Place the rod on the stake T. P., and noting that the bubble is still at the center, direct the telescope upon the rod and mark the point intersected by the cross wires. This height of rod is called a "Fore-sight," and is the vertical distance from the line of collimation down to the stake "T. P." Denote it by "b" and enter it opposite "Sta." T. P. in column "F. S." The observed "elevation" of the stake T. P. with reference to B. M. is (a — b). Enter this in the column "Elevation."

Move the transit to T. P. and set it up on line and close to the stake, so that the eyepiece shall be about one-half inch from the rod held vertically on T. P. Bring the telescope bubble to the middle of the tube and read the rod as before by looking backward through the telescope. The result (a′) is a back-sight on T. P. and is so entered in the record on the third line. Since the elevation of the T. P. is $a - b$ and the line of collimation is higher by a′, the new H. I. is $a - b + a'$, in column "H. I."

Sight forward through the telescope and read the rod held again on stake B. M. This reading, b′, is a fore-sight and is recorded opposite sta. B. M. in the fourth line and in column "F. S." It indicates that the stake B. M. is lower than the line of collimation by b′ and gives $a - b + a' - b'$ as the observed elevation of the stake B. M. If the line of collimation were horizontal this observed elevation of B. M. would be equal to the assumed elevation, zero. Conversely, if the observed elevation of B. M., $a - b + a' - b'$, is not equal to the assumed elevation, zero, the line of collimation was not horizontal, and the error $a - b + a' - b'$, was due to its inclination. If the value of the error is positive, the back-sights, a and a′, were too great and the line of collimation inclined downward; if negative, the back-sights, a and a′, were too small and the line of collimation inclined upward. The two sights, from B. M. to T. P. and from T. P. to B. M., were equal in length and in inclination. Therefore in the last sight from T. P. to B. M., the error was one-half of the total error, or $\frac{1}{2}(a - b + a' - b') = c$. Move the last mark on the rod at B. M., upward if c is positive and down if c is negative, over a distance equal to c. Direct the cross wires upon this new mark, and the line of collimation will be horizontal. With the adjusting screws of the level, make its axis horizontal (bring bubble again to the middle of the tube). The axis of the level tube and the line of collimation will then be parallel.

To test the adjustment, repeat the operation above described and if the new value of a—b+a′—b′ =0 the adjustment is complete.

In this method a slight error due to curvature of the earth's surface is introduced, but as this error amounts only to 0.001 ft. in 200 feet, and to 0.00025 ft. in 100 ft., it may be disregarded.

By analyzing this method it is seen that it consists of running a line of levels that closes on the starting point, and thus provides means of determining the error due to the inclination of the line of collimation. Since there are two courses of equal length, one-half the error belongs to each course, and this gives the correction "c," which must be applied to make the line of collimation horizontal.

EXAMPLE.

Sta.	B. S. +	H I.	F. S. —	Elev.
B. M.	3.256	3.256		0
T. P.			5.813	—2.557
T. P.	4.086	1.529		—2.557
B. M.			2.506	—0.977 ÷ 2 = —0.488

In this case the value of *c* is — 0.488 ft. and the last mark on the rod must be moved downward 0.488 ft. If the cross wires be made to bisect this new mark the line of collimation will be horizontal.

Second Method: On ground which is fairly level, set up and level the transit. Bring the bubble of the telescope level to the middle of its tube and clamp the telescope. At a distance of 100 ft. measured accurately from the center of the transit, drive a stake firmly and upon it hold a rod. Mark the point on the rod that is cut by the horizontal wire of the telescope. Turn the telescope 180 degrees on its vertical axis and at the same measured distance (100 ft.) in the new direction drive a second stake, and testing its height by the rod, continue driving until the mark on the rod is cut by the horizontal wire of the telescope. The tops of the stakes will then lie in the same horizontal plane, no matter what may have been the inclination of the line of collimation. Move the transit and set it up close to one of the stakes so that the eye end of the telescope shall be about one-half inch from the rod held on the stake. Looking into the object glass, sight backward through the telescope at the rod and mark the point of the rod that is seen at the center of the small field of view. Then place the rod on the other stake and looking forward through the telescope, bisect the same point with the cross wires. This will make the line of collimation horizontal because it has been made parallel to the line joining the tops of the stakes. A second trial should be made to check the first result. Having verified the horizontality of the line of collimation, adjust the telescope level by the screws that control it and bring its bubble to the middle of the tube. The axis of the level will then be parallel to the line of collimation.

Fifth Adjustment.—To cause the vernier of the vertical circle to read zero when the line of collimation is horizontal.

This adjustment should be made in connection with the fourth adjustment.

1st. When the line of collimation has been made horizontal by either of the methods of the fourth adjustment, then, without moving the telescope, loosen the screws that hold the vernier of the vertical circle and move the vernier until it reads zero on the limb. Fasten the vernier in this position and the adjustment is completed.

2d. When the vernier on the vertical circle is not adjustable this adjustment must be made by moving the line of collimation, that is, the cross wires, instead of the vernier. Therefore, when making the fourth adjustment, set the vertical circle to read zero and then make the line of collimation horizontal by adjusting the cross wires with the top and bottom adjusting screws. This may disturb the second adjustment and a test should be made to correct a possible lateral displacement of the cross wires.

3d. When the vertical arc is a full circle the following method may be used. Level the transit carefully and direct the telescope at a well defined point or mark, 300 to 500 feet distant and read the vernier of the vertical circle. Reverse and plunge the telescope and point it at the same mark. Read the vernier again and find the mean (one-half the sum) of the two readings. Adjust the vernier so that its reading shall be the mean of the two readings. It will then indicate the correct vertical angle and will read zero when the line of collimation is horizontal. If the vernier is not adjustable, turn the telescope till the reading of the vertical circle is the mean of the two readings, and then with the top and bottom adjusting screws bring the intersection of the cross wires again upon the selected point or mark. When this last method is used for the fifth adjustment it should precede the fourth adjustment, which is then made as follows: Set the vertical circle at zero. This makes the line of collimation horizontal. Bring the bubble to the middle of its tube by the adjusting screw. This makes the axis of the bubble tube horizontal and therefore parallel to the line of collimation.

REMARKS ON THE ADJUSTMENTS.

It cannot be assumed that the adjustments of the transit or of any instrument can be made with perfect accuracy, but when the adjustments are carefully made and tested, the errors that remain uncorrected are so small that they do not appreciably affect the results in ordinary surveying. When greater accuracy is required each observation should be repeated (with the instrument reversed if possible) and the mean of the two readings adopted. For the greatest attainable accuracy, the best of instruments must be used and the observations must be repeated, five, ten, or even twenty times, with alternate reversals of the instrument whenever reversing is applicable.

In the adjustments given no attention has been paid to making the line of collimation pass through the point of intersection of the horizontal and vertical axis, because this condition cannot be fulfilled at the same time with the others. If it does not pass exactly through this point the errors that arise are those of parallax only, and are so small as to be negligible in ordinary surveying.

When the second adjustment has been made it is impossible to make any other lateral adjustment of the cross wires without disturbing the second adjustment. Therefore the line of collimation cannot be made to intersect the vertical axis without sacrificing the second adjustment. It may, however, be made to intersect the horizontal axis without interfering with other adjustments. If it is desired to make this adjustment, proceed as follows: Set up the transit about 50 feet from a building or wall. Rest the object end of the telescope in a notch cut in the top of a board that has been set firmly in the ground. Sight through the telescope at the wall and signal an assistant to mark a pencil dot at the point intersected by the cross wires. Reverse and plunge the telescope and rest the object end again in the notch. Mark the point that is now intersected by the cross wires and draw a horizontal line midway between the two dots. With the top and bottom adjusting screws move the cross wires till their intersection falls upon the horizontal line. The line of collimation will then intersect the horizontal axis, as will be evident from an examination of the figure (Plate III, Fig. 2). O is the projection of the horizontal axis. C L D is the first position of the line of collimation. C′L D′ is its second position after reversing and plunging. P L O is its adjusted position passing through the point P midway in elevation between D and D′ and therefore passing also through O, the horizontal axis, midway between C and C′. The notch in the board holds the point L at the same place in both positions of the telescope.

If this adjustment is made the fifth adjustment must be made by adjusting the vernier of the vertical circle and not by moving the cross wires. If the vernier is not adjustable its error must be treated as an index error to be applied to all readings.

The object glass slide is not always made adjustable. When it is adjustable, the adjustment is carefully made by the manufacturer and the adjusting screws are covered by a ring to discourage ill-advised attempts at readjustment.

The adjustment of the eyepiece consists simply in moving it by its adjusting screws so that the intersection of the cross wires shall be in the center of the field of view.

Whenever the cross wires are adjusted, care must be taken to preserve the verticality of the vertical wire. To test this, sight at a plumb line.

EFFECTS OF FAULTY ADJUSTMENT.

(1.) If the vertical axis is not truly vertical the limb of the instrument, which is perpendicular to the axis, will not lie in a horizontal plane and the observed angles will not be the horizontal angles between the points observed. Neither will the vertical circle lie in a vertical plane, nor will a zero reading of this circle always indicate a horizontal position of the line of collimation. Observed vertical angles will therefore be in error.

(2.) If the line of collimation is not perpendicular to the horizontal axis it will not follow a vertical line through a given point when revolved on the horizontal axis. The horizontal angle between any two points on a plumb line is, of course, zero; but if the line of collimation be directed first at a high point, and then be plunged downward to a lower point, some change in azimuth will be found necessary to bring the cross wires again to the plumb line. This change is the error due to the inclination of the line of collimation. This same error would affect the observed horizontal angle between any two points having respectively the same angles of elevation as the two points considered. If two points have equal angles of elevation there would be no error from this cause in the observed horizontal angle between them.

The reading of vertical angles is only slightly affected by a small obliquity of the line of collimation.

(3) If the horizontal axis is not truly horizontal the error in horizontal angles is similar to that caused by obliquity of the line of collimation. The cross wires would not follow a plumb line, and their deviation from the plumb line in plunging from a high to a low point would be the error in the horizontal angle between those points, or between any two points having respectively the same angles of elevation as the points considered. If two points have equal angles of elevation there would be no error from this cause in the observed horizontal angle.

In reading vertical angles a slight error would be introduced because the plane in which the angle is measured would not be a vertical plane.

(4.) If the fourth adjustment is faulty, errors will be made when using the transit as a level. These errors are discussed in connection with the description of the level.

(5.) If the vernier (or index) of the vertical circle is not properly adjusted, the error of adjustment will appear in all single measurements of angles of elevation or depression. If not adjusted, the index error should be determined and applied to all readings of this circle, or the mean of a direct and reversed reading should be taken.

ATE II

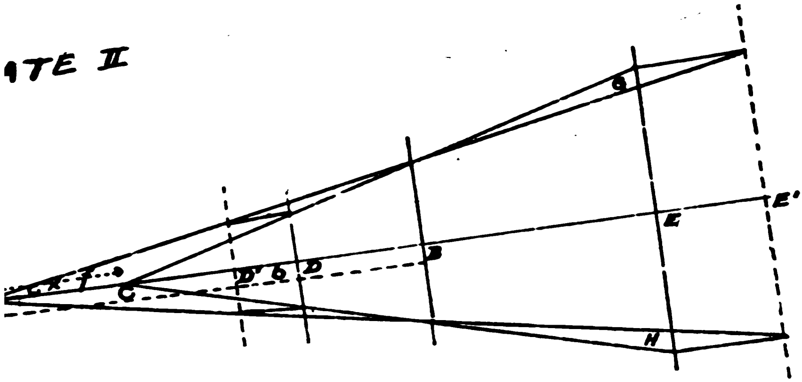


Fig 1

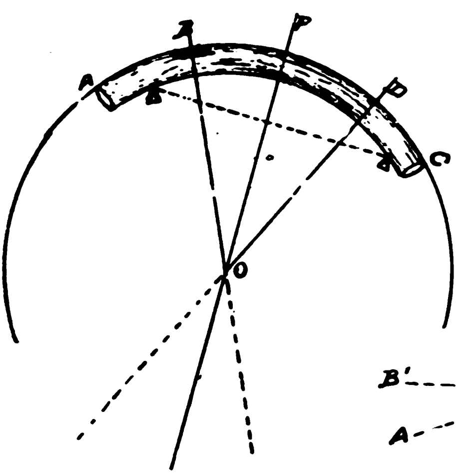


Fig. 2

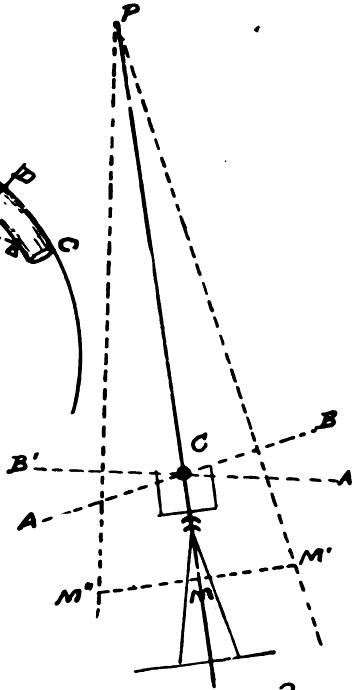


Fig. 3

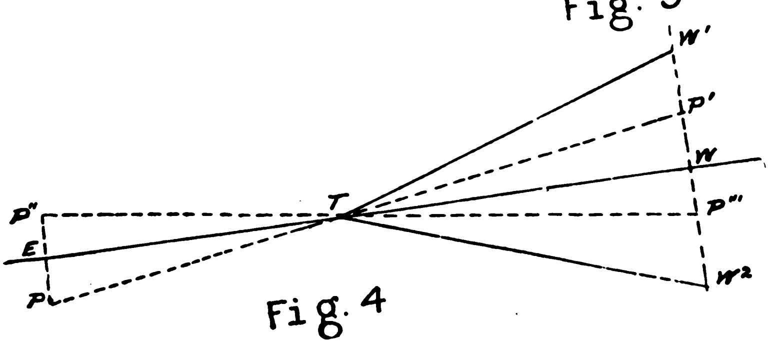


Fig. 4

CHAPTER V.

CARE AND USE OF THE TRANSIT.

When first opening the case containing a transit or other instrument note the position of the telescope and of the clamp screws, and also the points of contact on the bearings that support the instrument in its proper position. Always thereafter return the instrument to its case in the same position. Observe also whether the instrument is provided with plumb line, reading glass, adjusting pins, screw driver, oil can, and dust brush, each in its proper receptacle.

To assemble the transit, set up the tripod with the legs equally spread and screw on the head of the instrument, taking care not to cross the threads and not to use excessive force. Attach the plumb line.

In carrying the transit, unclamp the telescope and lower plate, but clamp the upper plate, and stop the compass needle. The unclamped parts permit the transit to adjust itself to shocks and knocks without undue strain; the clamping of the upper plate maintains the orientation that is often to be transferred from one station to another; and the stopping of the needle prevents wear of the pivot.

To *set up* the transit means to place it in position for use with its center over the station point and with its axis made vertical by means of the leveling screws. The station is usually marked by a tack in the top of a stake that is driven into the ground.

Any well defined point on or near the ground may be used.

Spread the legs of the tripod equally around the station point and adjust them so that the plates shall be nearly level and the plumb bob nearly touching the tack that marks the station. With the shifting center bring the point of the plumb bob over the center of the tack or other station mark, and, with the leveling screws, make the axis of the transit vertical. To accomplish this last object proceed as follows: Turn the head of the transit so that the plate levels shall be parallel respectively to the lines joining opposite leveling screws. With one pair of screws bring the bubble of the parallel level to its center by turning the screws simultaneously in opposite directions, that is, by moving the thumbs toward each other or away from each other. The bubble will follow the motion of the left thumb. With the other pair of leveling screws bring the other bubble to its central position. Repeat these operations alternately till both bubbles stand at the center of their tubes. The axis of the transit will then be vertical.

Setting up or *to set up* the transit includes all of the operations just described, and it will be so understood in the following pages.

To reverse is to turn the transit 180 degrees, or approximately 180 degrees, on its vertical axis.

To plunge is to turn the telescope through any indicated angle on its horizontal axis or trunnions.

When sighting through the telescope for the final pointing at any definite point, be careful not to touch the instrument at any part. The lightest touch of the fingers on the clamps or the contact of the cloth-ing against the tripod will cause an appreciable error in the reading of an angle.

Transits are built mainly of brass, bronze and wood. These are comparatively soft materials and will not stand hard usage. The application of any considerable force will wear or strip the threads of the screws, bend the screws or spindles, or otherwise disarrange the delicate mechanism, and ruin the instrument. The handicraft in the use of an instrument consists in applying the strength and skill of the fingers only. Never apply the strength or weight of the wrist, arm, or body.

The transit should be kept perfectly clean and dry. Dust and grit are the greatest enemies of the transit. Next to these comes water. Therefore, brush it with the camel's hair brush or wipe it dry with a soft cloth whenever it has been exposed to dust or moisture. The transit should be kept in its case when not in use, if only over night, or else the head of the instrument should be covered with the gossamer hood.

The care of the graduated circles deserves special attention. The lines are engraved on silver and filled with black wax. Through neglect the circles may become tarnished and dirty. An occasional rubbing with a chamois skin will keep them in good order. Dirt is easily removed by a cloth moistened with alcohol. If the graduations are worn and scratched by dust, grit, and neglect, the instrument should be sent to the makers for repairs. The use of emery, chalk, pumice, or other polishing powder or composition, on a graduated circle is an evidence of gross ignorance, and an end to the usefulness of the instrument.

The working parts of the transit should move smoothly and freely without catch or jerk, and without chafing or grating. A grinding sound indicates the presence of dust or grit in the bearings. When this occurs the instrument should be taken apart, cleaned with alcohol and oiled by rubbing with a well oiled rag, not by a squirt can. Only the best clock or typewriter oil should be used.

The transit should always be handled as if it were made of thin glass.

The principles and general methods involved in surveying are the same for all instruments and must be stated before proceeding with a description of the use of the transit.

Surveying is a time-consuming and expensive operation, and satisfactory progress cannot be made if the comfort and ease of the members of the party are considered. A surveying party should be composed of men who are willing to rough it, to work early and late in all kinds of weather, and to lend a hand at any labor that will expedite the work. In clearing out long lines in wooded country every man in the party becomes an axeman and does his share of the work. In building high stations every man becomes a carpenter. Whatever the work, each man bears his share, and there is no excuse for any member of the party to be idle at any time.

The plotting of the field sheets should keep pace with the field work. The evenings and the stormy days are utilized in reducing the notes, plotting the work, and in overhauling and cleaning the equipment. When a draughtsman is employed, he plots the final map as fast as the notes of the surveyor are furnished him, and when the field work is completed he should require only two or three additional days to finish the map, while the surveyor or chief of party prepares his report.

When the beginner first sets up an instrument and looks around over the country which he is required to survey, his task seems hopeless. There appears to be no definite "point" to which he can make a measurement, because all points seem alike indefinite. It is in the selection of the points to be determined that the skill, judgment and experience of the topographer are tested. The manipulation of the transit or other instrument is a mechanical operation readily performed after a little practice. But this operation is valueless unless the points observed are *critical points*.

For example, in surveying a quadrilateral field inclosed by straight fences on the four sides, let four points, at the middle of the four sides respectively, be determined as a, b, c and d. (Plate III, Fig. 3.) These points would give no information as to the shape of the field. But, let the four corners A, B, C and D be located by observations, and the shape and size of the field are fully determined. On ground with varying slopes, as shown in profile in Plate III, Fig. 4, the location of the points a, b, c, d and e would give no idea of the shape of the profile, but if the points A, B, C, D and E, where the slope changes, be located, the different slopes are fully determined.

Again, let the points a, b, c, d and e, (Plate IV, Fig. 1) on the 680 ft. contour be located; the line thus determined would be the dotted line a, b, c, etc.; but if the critical points, A, B, C, D, etc., be located and plotted, the curved line drawn through these points will represent with considerable accuracy the actual run of the contour.

A *critical point* is a point that is common to two or more intersecting lines or surfaces. On curved lines or surfaces, it is a point where the curvature is most pronounced or where the curvature begins or ends. Points at the corners of a house, at fence corners, at the junction of streams, at a bend in a road or stream or shore line, on the crest line of a ridge or bottom line of a valley, at the crest and foot of a slope, terrace, embankment or cut, at the salients and reëntrants of the border of a forest, at the beginning and end of curves on a railroad or road, etc., are critical points. If all such points be determined the lines and surfaces that join them are determined, and the map made by plotting these points and filling in the details by the usual conventional signs and symbols will be a correct representation of the features and incidents of the area considered. If the critical points are not determined, the shape, features and incidents of the ground will not be fully determined, and cannot be correctly represented on the map, no matter how many random points may be determined.

Note the water course, watershed, and change-of-slope lines and on these lines select the critical points. These and other critical points already described are the points to be determined in surveying.

TRANSIT AND STADIA SURVEY.

The transit with stadia attachment is a complete and universal surveying instrument. When set up at a known point and directed at a required point, the reading of the horizontal circle, vertical circle and stadia rod, give respectively the horizontal angle from the assumed standard direction, the vertical angle from the horizon, and the distance; that is, the three polar coördinates that fully determine the unknown point. The surveyor simply points the telescope and then reads and records what the instrument tells him.

PARTY.	EQUIPMENT.
One surveyor	Transit, note book, pencil, etc.
One rodman	Stadia rod. hatchet
One axeman	Axe and stakes.

This is the minimum, and is suitable only for a small survey. For an extended survey the party should consist of one surveyor, one recorder, one draughtsman, two rodmen and two axemen each having the necessary equipment. If the party is in camp there must be added one cook and one teamster with camp equipage, team, and wagon. Such a party will cost from \$20 to \$25 per day, and will cover daily from one-half square mile to one square mile, depending on the character of the country and on the accuracy and amount of detail required. The cost per square mile will therefore vary between \$20 and \$50 for an accurate topographical survey with the transit and stadia.

It requires an experienced surveyor to run such a party and keep every man employed all the time. The beginner will find it difficult to keep one rodman and one axeman busy, and is doing well if he covers one square mile in a week. It does not take long, however, to become an experienced surveyor, if the principles are well understood. The fussing, the false moves and the hesitancy of the first week will largely disappear in the second, and a month of steady field work will make a good surveyor of a man who tries. On the other hand, after years of experience there will still be new things to learn. Long practice and a knowledge of the higher mathematics are necessary for very accurate work.

In beginning the survey, if an established bench mark is accessible it should be used as the origin or starting point of the survey. (A bench mark is a definite point on a fixed and permanent object the elevation of which point with reference to a fixed or assumed datum plane has been determined. Refer to the subject of "Bench Marks.")

If no existing bench mark is available, select a suitable point and make it a bench mark by assuming its elevation. Set up the transit near the bench mark, selecting as the initial station (□ 0) a point from which a good view may be obtained to surrounding critical points including the bench mark.

Stations are usually designated by the following symbols: □ represents a transit or plane table station, O a compass station, △ a triangulation station. A dot is usually placed in the center of each station symbol.

To orient the transit at the initial station unclamp the upper plate and turn it so that vernier A shall read zero. Clamp in this position and set accurately by the tangent screw, using a magnifying glass to read the vernier, and place its index exactly at the zero mark of the limb.

Unstop the needle and turn the head of the instrument so that the north end of the needle shall read zero at the N point of the compass circle. Clamp the lower plate and insure an accurate zero reading of the needle by means of the lower tangent screw.

The horizontal circle is now oriented and fixed in position by its clamp. If the upper clamp be loosened and the alidade turned so that the telescope shall point at any desired object, the vernier (A) and the needle will each indicate the horizontal angle between the north point and the point observed. The compass circle should be graduated to the left from zero to 360 degrees, and the lower limb to the right from zero to 360 degrees. The readings of the compass and of the index to the lower limb will then be the same for all angles, and the compass will serve as a check on the readings of the index. Beginners should always note the compass reading and see that it verifies the reading of vernier A. This verification extends only to the degrees and quarter degrees because the compass cannot be read as accurately as the vernier, but it is in the degrees that errors, or rather mistakes, are often made.

The reading of the vertical circle gives the angle of elevation or depression to the point observed, and the reading of the stadia rod held at the point observed gives a distance which by means of the formulæ (page 85) or by the table or diagram referred to on page 86 may be reduced to the horizontal distance and difference of elevation or vertical distance.

When the transit is set up, the rodman holds the stadia rod on the ground vertically and close to the telescope, and places a rubber band around the rod at height of the telescope trunnions. This marks on the rod the point to be observed when reading vertical angles. The surveyor notes the height of this mark on the rod and enters it in the record as Height of Instrument (H. I.), which here means height above the ground, not above datum plane. This record enables the surveyor to see that the rubber band is not displaced during subsequent observations from this station.

The rodman holds the rod first at the bench mark and then, in succession, at other points to which he is directed by the transitman. He must always hold the rod vertical with its center over the selected point.

When the rod is properly placed at a selected point, the surveyor directs the telescope at the rod and bisects the rubber band with the cross wires, using only the clamp and tangent of the vernier plate and the clamp and tangent of the telescope. The lower clamp must not be touched after orientation. He reads vernier A, and the vernier of the vertical circle, and the compass, and records the readings. Then sighting again at the rod he brings the lower stadia wire to the nearest 100-foot division, (or to the "c + f" mark) and counts the hundreds, tens and units of the rod divisions up to the point cut by the upper stadia wire. This is entered in the record as the stadia reading.

When all the critical points that are accessible from station 0 (□ 0) have been observed a new station (□ 1) is established. A point is selected for this station that will advance the survey in the desired direction and will afford good sights to the new points that must be determined. The readings on the rod to this station are the same as those made to determine other points, but they are made with greater care and deliberation. Any error made in the location of a station affects all subsequent determinations.

When the readings on the new station (□ 1) are completed, unclamp the lower plate (leaving the vernier plate clamped), unclamp the telescope, and stop the needle. Carry the transit to station 1 and set it up; free the needle, plunge the telescope on the horizontal axis and cause the rodman to set the rubber band at the new height of instrument. Send the rod back to station 0, and with the telescope plunged sight at the rod and bisect the rubber band, using only the lower clamp and tangent and the telescope clamp and tangent. Do not touch the clamp or tangent of the vernier plate. Since vernier A has remained set at the forward pointing from □0 to □1 and the telescope now points back from □1 to □0, the line of collimation is parallel to its former position, and so also is the zero line of the horizontal circle parallel to its former position. This backsight from □1 to □0 therefore orients the transit at □1 with every line on the lower plate parallel to its former position at □0 and the azimuth readings will have their origin or zero at the north point as before. Check the orientation by reading the needle. The compass reading should coincide with the reading of vernier A.

Plunge the telescope back to its normal position, unclamp the vernier plate and proceed with the observations on the critical points that can be located from □1. The last observation at □1 is the one which locates □2, and the transit is then carried to □2 and oriented by a backsight on □1 with telescope plunged. The accessible critical points are then located from □2 as described for □0 and □1. The foregoing operations are repeated at successive stations until the required area or line has been covered.

Orientation by a backsight with telescope *plunged* is affected by an error in the adjustment of the line of collimation. It is therefore considered better to point the telescope backward by reversing it 180° instead of plunging it. To do this, set vernier A at a reading that differs by 180° from the last forward reading and point at the last preceding station. The instrument is now oriented and the work may proceed as described.

The foresights and backsights between stations along the main traverse must be made with special care and accuracy since they serve to refer all determinations back to the origin, and any error is transmitted through all subsequent observations. The accuracy of the traverse as a whole may be checked by running the traverse in a circuit and closing on the initial station, or by running the traverse between two points that have been accurately determined by some other method, as by triangulation. If no such check is applied, there can be no certainty as to the accuracy of the survey, and large errors, or even mistakes, may be made without detection. If the traverse closes on a known point, whether it be the initial point or a triangulation point, any error or mistake will be detected. Legitimate errors within the limits of error imposed by the requirements of the work may be adjusted, but mistakes can be corrected only by running the traverse over again to the point where the mistake was made. If the compass is constantly used as a check on the azimuth readings, there should be no mistakes in the determination of azimuths, but there is no constant check on the stadia readings except care and attention on the part of the surveyor.

A mistake is a wrong reading of a vernier or of a rod, a pointing made at the wrong point, a faulty orientation of the instrument, or a false entry or omission in the record.

Mistakes are avoidable.

Errors are small deviations from accuracy due to the imperfections of the instrument and of the human faculties.

Errors are unavoidable.

SUMMARY OF OPERATIONS.

1. Set up the transit at □ 0, and orient by the needle or by pointing in a known direction.
2. Backsight on bench mark.
3. Sideshots on the accessible critical points.
4. Foresight on □ 1.
5. Move to □ 1 and set up transit.
6. Orient by backsight on □ 0.
7. Side shots on required points.
8. Foresight on □ 2.
9. Repeat 5, 6, 7 and 8 at successive stations.
10. Close on the initial station or on a triangulation station.

THE RECORD.

The notes taken at each station constitute a separate section of the Record. The heading of each section (see example) gives the numeral of the station occupied, the height of instrument (H. I.) at the station, and a blank space for the elevation of the station point. Below this heading enter, 1st, the readings of the backsight to the previous station; 2d, the readings of all side shots taken at this station; 3d, the readings of the foresight to the next station.

The readings for each sight or sideshot include the azimuth; the bearing (used as a check only); the vertical angle, + for angles above the horizontal and — for angles below the horizontal; and the stadia distance.

Example: (*First page or title page of each note book.*)

SURVEY OF

FORT LEAVENWORTH, KANSAS

AND VICINITY

Made under the direction of

....

by

...., Surveyor, in charge of party.
...., Recorder.
...., Draughtsman.
....,
...., } Rodmen.
....,
...., } Axemen.

Azimuth and bearings are from the magnetic north to the right, zero to 360 degrees.*

Book No. .... (................ books in all),

March 3, 1906, to .... 190......

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*The record should show whether azimuth and bearings are magnetic or true and in what direction they have been read.

(LEFT HAND PAGES OF RECORD.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Point Observed.	Azimuth Vern. A.	Bearing Compass	Vertical Angle	Stadia	Horiz. Distance.	Vertical Distance.	Eleva- tion	Remarks.
At	*Sta.*	*0,*	*H.*	*I. =*	*453.*		*Elev.*	=
	° ′	°	° ′					
BM 1	231–46	231¾	—3–21	489			732.41	BM No. 65 US Sur'y Mo Riv
C. P.								
	etc.			etc.			etc.	
Sta. 1	35–24	35½	+5–52	728				Top of Turtle Hill
At	*Sta.*	*1,*	*H.*	*I. =*	*445.*		*Elev.*	=
Sta. 0	215–24	215¼	—5–20	726				Backsight
C. P.								Fence Corner
C. P.								S. E. Corner House
C. P.								S. W. Corner House
	etc.		etc.				etc.	
Sta. 2								Near Forks of Stream

The note books used in a survey should also be named and numbered on the cover. These books are not the personal property of the surveyor, but pertain to the office or person for whom the survey is made. The note book, map and report constitute the complete record of the survey.

The field record begins with a statement describing the method of reading angles, and the orientation adopted.

The notes taken at the initial station have the heading, "At Sta. 0, H. I. =453, Elevation ——," and under this heading there are entered all the readings taken at this station. The first sight is taken on a bench mark, supposed in this case to have been established by a government survey. In column (1) write the designation of the point observed (B. M. 1); in column (2) the *azimuth*, or reading of ver. A (231°—46′); in column (3) the *bearing*, or reading of the needle (231¾°); in column (4) the *vertical angle*, or reading of the vertical circle (—3° 21′); in column (5) the *stadia distance*, or reading of the stadia rod (489). Columns (6), (7) and (8) are the reductions to horizontal distance, vertical distance, and elevations above datum, respectively, deduced from columns (4) and (5) and filled in after the day's field work is completed. Column (9) contains definite descriptions of points observed, and any other needed remarks of an explanatory nature.

Similar record is made of readings on other points, and the last observation from Sta. 0, is made on Sta. 1 to determine a new point from which further observations may be taken.

The right hand page of the record should contain a free-hand sketch made in the field as the work progresses, and showing all points located by the observations and the objects and features that are thus located.

The horizontal angles may be plotted direct from the records, but the vertical angles and stadia readings cannot be plotted until they have been reduced to horizontal distance and vertical distance. The horizontal distance can then be plotted or laid off to scale, from the station point, in the direction given by the azimuth, to the required position of the point observed; and the elevation of this point may be shown by writing, at its plotted position, the number that indicates its vertical distance in feet above the datum plane. The map position of the point is thus fully expressed.

The methods of reducing inclined stadia readings to horizontal distance and vertical distance by formulæ or tables or diagrams have been fully explained under the head of "The Stadia"

Reduction by means of the formulæ is the most accurate method, but is a slow and tedious operation seldom used. The tables are sufficiently accurate, and their use involves only a simple multiplication of the stadia reading by the tabular numbers that correspond to the vertical angle. The diagram gives the horizontal and vertical distances without computation and with an accuracy equal to that attainable in reading the stadia or in plotting the results. It may be used, therefore, for all sideshot reductions. For reductions of sights on the main traverse, if greater accuracy is required, the table should be used.

In *plotting the survey* the main traverse and the sideshots are considered separately. The traverse is plotted first, and when all the traverse stations are correctly located on the paper the sideshots taken from each station are plotted. The points so determined serve as the basis for drawing the lines and conventional signs that represent in proper relative positions, the objects, features and incidents that have been located.

TO PLOT THE TRAVERSE.

A protractor is a thin flat piece of metal, paper, celluloid, wood, or other material, near the edge of which are radiating lines spaced at degree or half-degree intervals. The center point from which these lines radiate is also marked, and the degree marks are numbered usually at 10° intervals to facilitate counting.

To plot any angle at a given point on a given line, place the center mark of the protractor at the given point and the zero line of the protractor on the given line. Count the degree marks in the desired direction to include the number of degrees in the given angle and make a dot on the paper at the point so determined. A line drawn through the center point and the dot so marked will make with the given line the given angle.

In plotting azimuths or bearings, the given point is the station point on the paper and the given line is the meridian line or other assumed standard direction line drawn through the station point

If not already plotted, assume a point on the paper to represent the initial point of the traverse, or station zero. Write its elevation and draw through it a meridian line. With the protractor lay off the azimuth of station 1, and draw the line representing the first course. With the working scale lay off the horizontal distance to station 1. At station 1 so determined write the elevation and draw a meridian. Proceed in the same manner from station 1 to station 2, from 2 to 3, and so on to the end of the traverse.

To plot the sideshots, set the protractor again at station 0, and lay off all the azimuths and distances taken as sideshots from that station. Mark the determined points, write their elevations, and draw the objects thus located. Proceed in the same manner at all the other stations. On the lines that join the plotted critical points, interpolate contour points and draw contours through points having the same reference. Plot in the same manner all the traverses of the survey.

To finish the map, ink the drawing and fill in, with conventional signs, the grass land, woods, marshes, cultivated ground, etc.; draw title, legend, scale, true and magnetic meridians and border. Erase all pencil lines and clean with sponge-rubber or bread crumb.

When the traverse begins and ends at known points or accurately determined points, as in the case of a closed traverse or of one that runs from one triangulation point to another, a check is furnished by means of which the errors made in traversing may be adjusted. If the traverse is correct, the plotted, or the computed position of the final station should coincide with the known position of that station. If these two points do not coincide, the line joining them is the error of closure and, in the contrary sense, is the correction that must be made in the plotted or computed position of the final station. This correction is used as a basis of comparison for determining the corrected positions of all other stations of the traverse under the assumption that the error is accumulative and proportional to the distances of the stations along the traverse from the origin. This assumption is not supposed to be true, but in the absence of any positive evidence to the contrary it is supposed to give positions that are probably more nearly correct than any other positions that can be assigned.

If a traverse be run between two known points A and B, of which *a* (*253*) and *b* (*291*) are the plotted positions, and if the line *a*, *1*, *2*, *3*, *b'* (Plate V, Fig. 1,) represents the plotted position of the traverse, [*b'*(*287*) representing the point B as determined by the traverse] the error *bb'* (-*4*) in the position of *b'* is due to the accumulated errors made in running the traverse. The position of *b'*, (*287*) may be corrected since its true position, *b* (*291*) is known, and the positions of the other stations may be adjusted to conform to the correction *bb'* (+*4*) applied at *b'*.

Each plotted station must be moved in a direction parallel to the line *b'b*, over a distance, *1-1'*, *2-2'* or *3-3'* respectively, that is proportional to the distance of the station from the origin *a*. The known ratio is the error of closure divided by the total length of traverse, and this multiplied by the distance of each station from the origin gives the corresponding correction.

The operations of adjustment may be tabulated as follows:

(1) STATION	(2) LENGTH OF COURSE	(3) DISTANCE FROM ORIGIN	(4) HOR. CORR.	(5) VERT. CORR.	(6) OBS. ELEV.	(7) COR. ELEV.
a	0	0	0	0	253	253.0
1	428	428	1.7	+ 1.1	264	265.1
2	410	838	3.3	+ 2.1	278	280.1
3	540	1378	5.5	+ 3.4	282	285.4
b′	196	1574	6.2	+ 4.0	287	291.0

$$\text{Hor. Ratio} = \frac{6.2}{1574} = .004. \quad \text{Vert. Ratio} = \frac{4}{1574} = .0025.$$

Through the stations draw lines parallel to *b′b* and on them lay off (column 5) 1.7 ft. at ☐ 1, 3.3 ft. at ☐ 2, 5.4 ft. at ☐ 3, giving the corrected stations, 1′, 2′ and 3′. At these new station points write the corrected elevations (column 7). When the adjustment is completed, erase the original traverse and adopt the new one.

When a traverse forms a circuit and closes on the initial station, the same method of adjustment is applicable.

When a survey is made up of a number of independent traverses each closing on a known point, the errors of each traverse may be adjusted and will not be carried to the next; but if the entire survey be made by a continuous traverse which does not close on any known point, there is no opportunity for adjustment, and no check on the accuracy of the work. A traverse should not run more than three miles without closing on a known point, and it is better to have check points at shorter intervals when practicable.

In the foregoing method of adjustment the horizontal error of closure may be due as much to errors in plotting as to errors in measuring the angles and distances. A better method of adjusting and plotting a traverse is the following:

METHOD BY LATITUDES AND DEPARTURES.

The polar coördinates given by the transit and stadia for each course are first reduced to the rectangular coördinates. The courses are considered consecutively in a forward direction along the traverse.

The vertical coördinate or difference of elevation for each course is given by the reduction of the stadia and vertical angle readings. The result is positive if upward and negative if downward. This reduction gives also the horizontal length of the course as already explained.

The latitude of a course is the component of its horizontal length in a north or south direction, and is equal to the horizontal length of the course multiplied by the cosine of the azimuth. The sign of the cosine determines the sign of the latitude.

The departure of a course is the component of its length in an east or west direction, and is equal to the horizontal distance multiplied by the sine of the azimuth. The sign of the sine determines the sign of the departure.

The azimuth is assumed to be measured from the north point to the right, zero to 360 degrees.

The signs of latitudes and departures are indicated in Plate V, Fig. 2. Let O be the origin or station occupied for any course and let the radius of the circle be the horizontal length of the course.

If the course O a lies in the first quadrant the cosine and sine of the azimuth are positive and therefore the latitude ($O a_1$) and the departure ($O a_2$) are positive.

In the second quadrant the cosine is negative and the sine is positive; therefore the latitude ($O b_1$) is negative and the departure ($O b_2$) is positive.

In the third quadrant the cosine and the sine are both negative, therefore the latitude ($O c_1$) and the departure ($O c_2$) are both negative.

In the fourth quadrant the cosine is positive and the sine is negative, therefore the latitude ($O d_1$) is positive and the departure ($O d_2$) is negative.

A positive latitude is also called a *northing* and a negative latitude, a *southing*. A positive departure is called an *easting* and a negative departure, a *westing*.

The algebraic sum of all the successive differences of elevation on all the courses of a traverse is the difference of elevation from the initial to the final station of the traverse. Thus if the traverse be shown in profile as in Plate V, Fig. 3, the algebraic sum of the differences of elevation is $+20-9-16+8+22=+25$, indicating that the point *b* is 25 feet higher than the point *a*. But if the point *b* is known to be 27 ft. higher than *a*, the total vertical error in the traverse is -2 feet, and the correction for each course is assumed to be proportional to the length of the course, with the total correction, $+2$ ft., divided by the total length of traverse, 1094 ft., as the known ratio, $\frac{2}{1094}=.0018$. Multiplying this by the several lengths of courses gives +0 47, +0.53, +0.39, +0.35, +0.23, as the corrections to be applied to +20, —9, —16, +8, and +22, respectively, to give the corrected differences of elevation, +20.47, —8.47, —15.61, +8.35, +22.23, the algebraic sum of which numbers is 27.00 as it should be.

In like manner the latitudes and departures are adjusted, for—

The algebraic sum of the latitudes of all the courses should be equal to the latitude of the traverse, and—

The algebraic sum of the departures of all the courses should be equal to the departure of the traverse.

If the traverse runs from one known point to another its total latitude and total departure are known and are used in adjusting the latitudes and departures of the several courses of the traverse.

In the particular case of a traverse that closes on the initial station, the algebraic sums of the differences of elevation, and of the latitudes, and of the departures, should each be equal to zero.

Let the line a, 1, 2, 3, b, Plate VI, Fig. 1, represent a traverse running from A to B. The departures of the several courses are respectively +a 1, +1 2, —2 3, +3 b, and their algebraic sum is the departure of the traverse, +a b.

The latitudes of the several courses are respectively —a′ 1′, +1′ 2′, +2′ 3′, +3′ b′, and their algebraic sum is the latitude of the traverse, +a′ b′.

If the sum of the departures is not equal to the known departure of the traverse, the difference is the error of closure of the departures and is distributed among the sevaral departures in proportion to the lengths of the corresponding courses.

Likewise, if the sum of the latitudes is not equal to the known latitude of the traverse, the difference is the error of closure in latitude and is distributed among the several latitudes in proportion to the lengths of the corresponding courses. Thus:

Course.	Horizontal Length.	Departure.	Corrections.	Corrected Departures	Departures from A.
a, 1	360	+ 321	— 1.40	+ 319.59	+ 319.59
1, 2	412	+ 346	— 1.61	+ 344 39	+ 663.98
2, 3	196	— 87	— 0.76	— 87.76	+ 576.22
3 b	310	+ 291	— 1.21	+ 289.79	+ 866.00
a, b	1278	+ 871	— 5.00	+ 866.00	+ 866.00

Known departure = + 866
Closing correction = — 5 Ratio = $-\frac{5}{1278}$ = — .0039

This ratio multiplied by the several horizontal lengths gives the corrections which, applied to the departures, give the corrected departures, whose continued sum gives the departures from A.

In the same manner adjust the latitudes and compute distances from an origin a′ with attention to signs.

In the same manner adjust the differences of elevation, and compute the elevations above datum.

To plot the traverse, draw on the paper an east and west line O Y (Plate VI, Fig. 1), and assume any point as a_1 as the origin of departures. From this point lay off the computed departures to 1_1, 2_1, 3_1, and b_1, and at these points erect perpendiculars. Draw a north and south line O X and assume any point as a′ as an origin of latitudes. From this point lay off the computed latitudes to 1′, 2′, 3′ and b′ and at these points erect perpendiculars. The corresponding perpendiculars intersect in the points a, 1, 2, 3, and b, and these points are the plotted positions of the traverse stations. At each station point write the computed elevation of the station.

This method of plotting is facilitated by ruling the paper in squares with sides representing convenient units of measure. This furnishes plotting scales in both directions by means of which the latitudes and departures are quickly plotted.

The advantages of this method of adjusting and plotting a survey are the following:

1st. The adjustment of errors is not affected by errors made in plotting with a protractor.

2d. The latitudes and departures may be computed and summed up in a short time, and errors or mistakes may be detected while there is a chance to correct them. If mistakes are not detected until the draughtsman plots the notes it will usually be impracticable to send the field party back to locate and correct the mistakes.

3d. Plotting by rectangular coördinates is more accurate than plotting with the protractor.

4th. The area included within a closed traverse or within any boundary that has been determined by sideshots from a traverse may be readily and accurately computed without plotting the survey, as will now be explained.

DETERMINATION OF AREAS BY MEANS OF DOUBLE MERIDIAN DISTANCES OR BY DOUBLE LATITUDES (PLATE VI, FIG. 2).

The meridian distance of a point is its distance from an assumed meridian measured on a parallel of latitude, as 1′ 1 (Plate VI, Fig. 2).

The double meridian distance of a straight line is the sum of the meridian distances of its extremities; thus 1′ 1+2′ 2 (Plate VI, Fig. 2) is the double meridian distance of the line 1, 2.

The double meridian distance of a course multiplied by the latitude of the course is twice the area included between the course and the meridian, and between the parallels of latitude drawn through the ends of the course; thus (0′ 0+1′ 1) ×0′ 1′ is twice the area 0, 1, 1′ 0′. The meridian is so assumed that all meridian distances are positive. Therefore, this double area is positive when the latitude of the course is positive, and negative when the latitude is negative.

Rule: Find the double area for each course of the closed traverse. Find the algebraic sum of these double areas and divide by two. The result will be the area inclosed by the traverse. The sign of the result is immaterial since only its numerical value is required.

Explanation: In (Plate VI, Fig. 2) take the sum of the positive areas 1, 2, 2′ 1′; 2, 3, 3′, 2′; 3, 4, 4′, 3′; 4, 5, 5′, 4′; and subtract the sum of the negative areas 0, 1, 1′, 0′; 5, 6, 6′, 5′; 6, 7, 7′, 6′. The result is the area 0, 1, 2, 3, 4, 5, 6, 7. Double areas are used only for convenience, as it is easier to divide the sum of all the areas by two than to divide each double area by two and then find the sum.

The result will be positive when the traverse runs contra-clockwise and negative when the traverse runs in clockwise direction, but as before stated the sign of the result is immaterial.

The same result is obtained by multiplying the double latitude distance of each course by the departure of the course to get the double areas, the signs being determined by the signs of the departures. One-half of the algebraic sum of the double areas gives the required area enclosed by the traverse.

It is often impracticable, on account of fences and other obstructions, to run a traverse on the boundary line of an estate or farm, but the traverse can be run near the boundary with sideshots to the corner points. The rectangular coördinates of the corner points may be computed as already explained and the area may then be determined by the method of double meridian or double latitude distances.

FORM FOR COMPUTATIONS AND REDUCTIONS OF A TRAVERSE.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
COURSE AND STATION	Az.	Hor. Dist.	Vert. Dist.	Vert. Corr.	Corr. Vert. Dist.	(Z) Elev.	Lat.	Lat. Corr.	Corr. Lat.	(Y) Lat. Dist.	Dep.	Dep. Corr.	Corr. Dep.	(X) Mer. Dist.	Doub. Mer. Dist.	Doub. Area. +	Doub. Area. −
B. M.																	
0																	
1																	
2																	
3																	
etc.																	
B. M.																	
Sum		=															
Known	value	=															
Error		=															
Ratio		=															

Columns 1, 2, 3 and 4 are copied from the reduced notes of the survey, omitting the side shots. Columns 8 and 12 are derived from 2 and 3. Find the sum of the horizontal distances (3) and the algebraic sums of the vertical distances (4), the latitudes (8), and the departures (12). If the traverse closes on a known point the total vertical distance, total latitude and total departure will be known and may be written in the corresponding columns opposite "known value." In a closed traverse the "known value" in each case is zero. The differences between the sums and the known values are "errors," and each error (4, 8 and 12) divided by the total horizontal distance (sum, column 3) gives the "ratio" of error.

Multiply the horizontal distances (column 3) by the vertical ratio to get the vertical corrections (column 5). Apply these corrections with proper signs to get correct vertical distances (column 6). The continued sums of correct vertical distances, beginning with elevation of B. M. gives the required elevations in column 7.

Multiply the horizontal distances (column 3) by the latitude ratio to get the latitude corrections (column 9). Apply these corrections with proper signs to the latitudes to get correct latitudes, (column 10). The continued sums of the correct latitudes, beginning with any assumed latitude for the initial station (0) give the "latitude distances" in column 11.

Multiply the horizontal distances (column 3) by the departure ratio to get the departure corrections (column 13). Apply these corrections with proper signs to the departures to get correct departures (column 14). The continued sums of the correct departures beginning with any assumed departure for the initial station (o) gives the "meridian distances" in column 15.

The latitude distance and meridian distance assumed for the initial station (o) should be such that all the latitude distances and all the meridian distances shall be positive.

The arithmetical results may be verified by summing up algebraically columns 6, 10 and 14. The results should be equal to the "known values."

Columns 11 and 15 give the rectangular coördinates to be used in plotting the traverse.

The last two columns (16 and 17) are used only when it is desired to find the area included within a closed traverse. The double meridian distances (column 16) are the sums of the meridian distances taken in consecutive pairs, thus o+1, 1+2, 2+3, etc. The double areas (column 17) are found by multiplying the double meridian distances by the corresponding latitudes (column 10). One-half the algebraic sum of the double areas is the area required.

PLATE III.

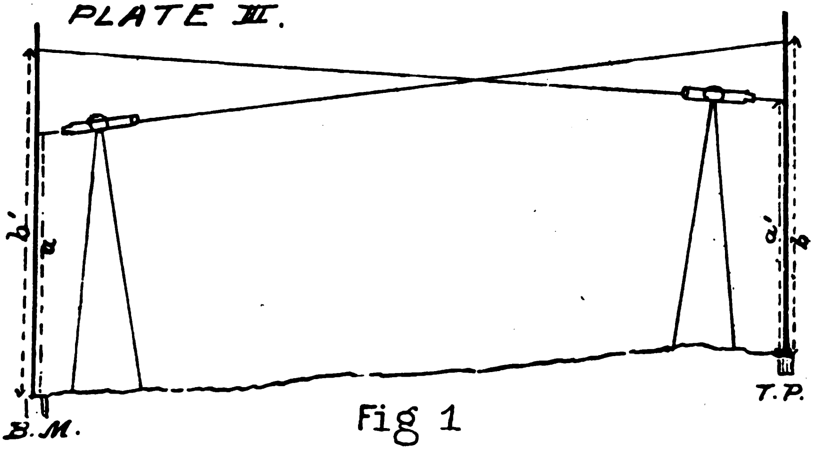


Fig 1

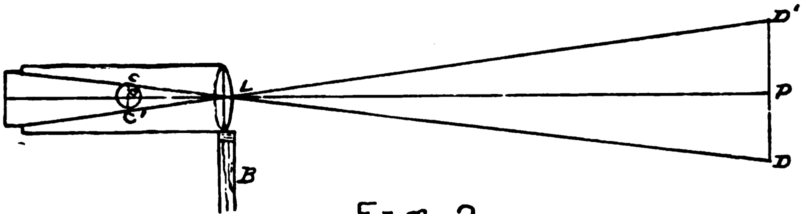


Fig. 2

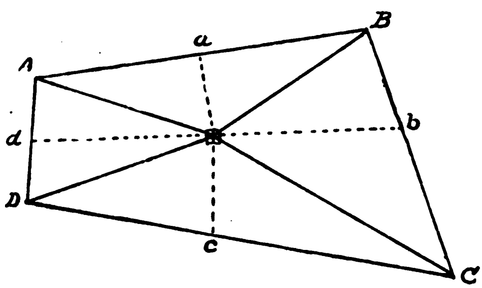


Fig. 3

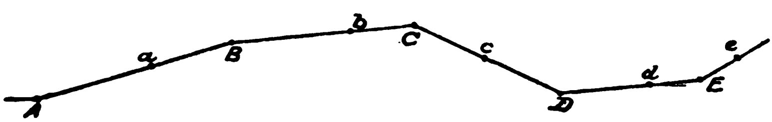


Fig. 4

PLATE IV

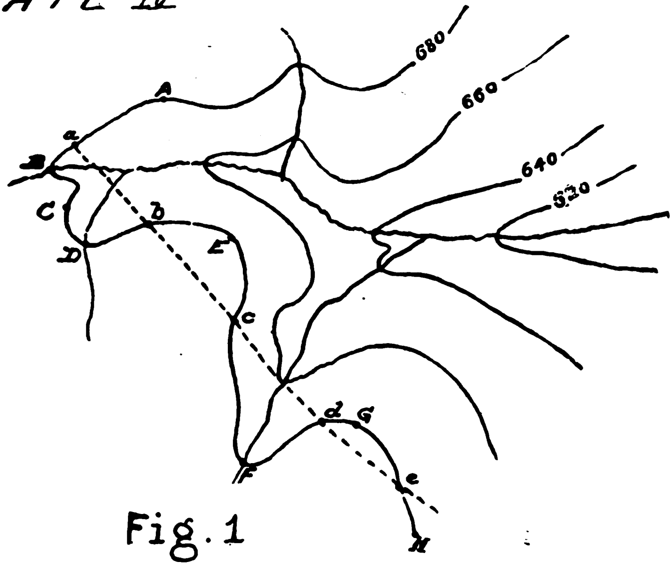


Fig. 1

PLATE V.

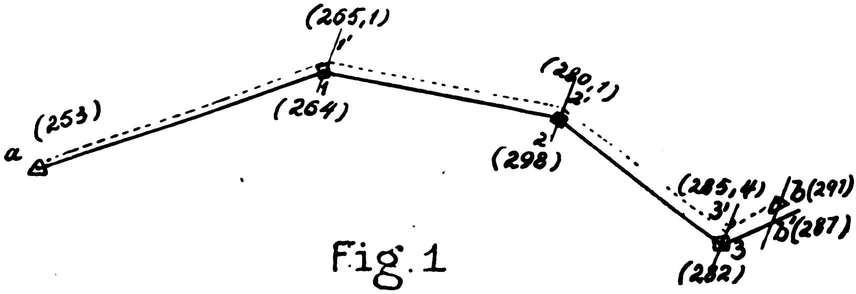


Fig. 1

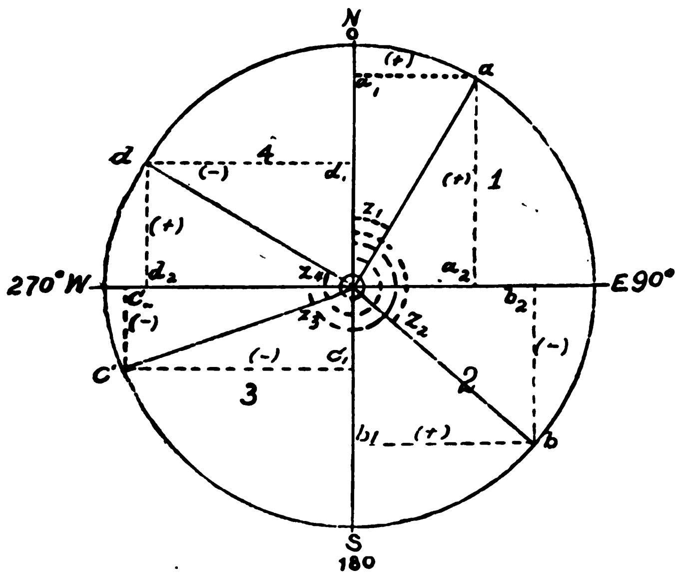


Fig. 2

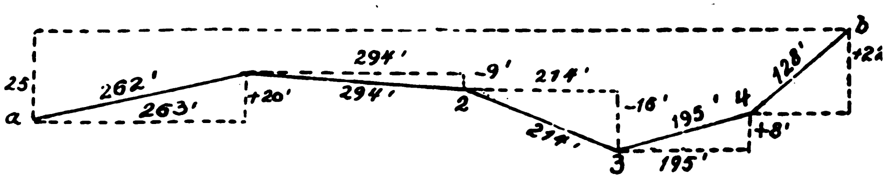


Fig. 3

PLATE VI

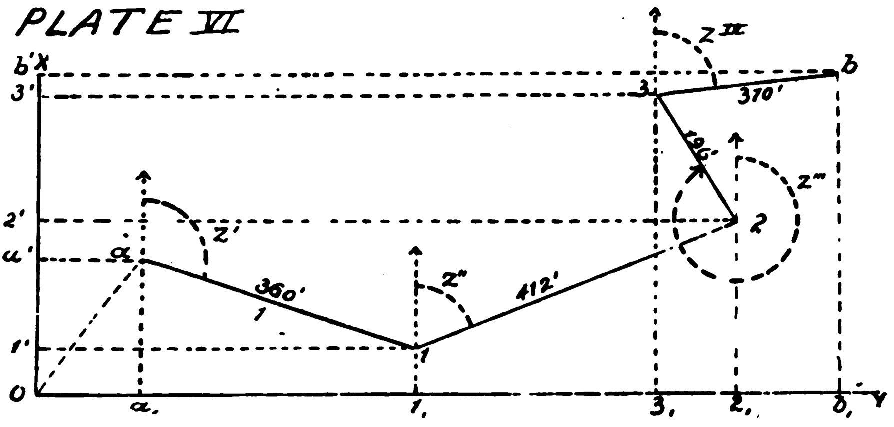


Fig. 1

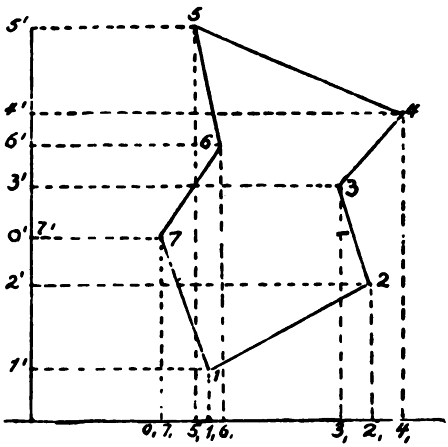


Fig. 2

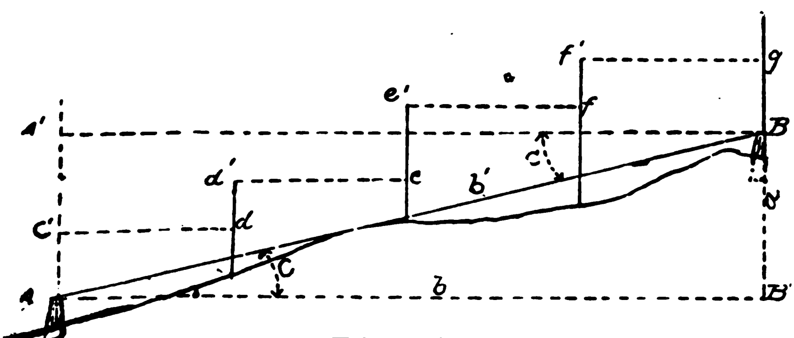


Fig. 3

CHAPTER VI.

THE COMPASS.

THE MAGNETIC NEEDLE.

If a straight bar of steel be magnetized and balanced horizontally upon a sharp-pointed pivot, with the least possible friction it will at any place assume a fixed position in the magnetic meridian through the place and will therefore determine a direction line, which may be taken as the standard for measuring other directions.

A thin bar of steel so magnetized and balanced is called a magnetic needle. It furnishes the only available means of determining the magnetic meridian at any place, and the magnetic meridian must therefore be defined as a line that coincides in direction with the magnetic needle at the place considered.

The directions indicated by the needle at different places on the earth's surface converge approximately toward the magnetic poles of the earth. These poles do not coincide with the poles of the earth's axis, and therefore the magnetic and true meridians do not generally coincide.

The angle between the magnetic and true meridians at any place is called the declination of the needle for that place.

There is a certain line on the earth's surface at all points of which the magnetic and true meridians coincide, and the declination is zero. This line is called the *agonic line* (without angle). In America it now passes through Michigan and South Carolina and intervening States, and is very slowly moving westward.

Conceive the agonic line to divide the earth roughly into two hemispheres. In the eastern hemisphere, thus formed, the north end of the needle inclines to the west of the true meridians and the declinations are therefore west, varying from 0° to 180°. In the western hemisphere, so formed, the north end of the needle inclines to the east of the true meridians and the declinations are east, varying from 0° to 180°. The declination becomes 180° only in the regions between the true poles and the magnetic poles.

The lines of magnetic force do not lie parallel to the earth's surface, but dip downward, to the north in the northern hemisphere, and to the south in the southern hemisphere. A needle hung at its center of gravity would therefore assume a position inclined to the horizontal. To counteract this tendency the needle is counterpoised, usually by a light coil of fine wire on the end that tends to rise.

Lines on the earth's surface at all points of which the declinations of the needle are equal are called *isogonic lines.* These lines, like the agonic line, are in America moving slowly westward. Judging by the results of observations extending over several centuries, it is predicted that in time this westward motion will cease and the isogonic lines will move east again.

These motions are attributed to a secular (long period) change in the positions of the magnetic poles, and the north magnetic pole is supposed to be moving toward the west in the region north of America. Consequently in America the north end of the needle is moving slowly toward the west following the motion of the magnetic pole; that is, west declinations are increasing and east declinations are decreasing. On the other side of the earth opposite conditions obtain, and there is an intermediate region, presumably that toward or from which the magnetic pole is moving, in which there is no change in declination.

This slow variation of the needle as it follows the moving magnetic pole is called the *secular variation* of the needle. The rate of this variation has been determined in different localities in minutes of arc per year, so that, if the declination be determined at any place and time, the declination at other times may be deduced. It is best, however, not to rely on computed declinations, but rather to determine it by actual observation at the place where and time when it is wanted. Besides the secular variation there is also a *daily variation* of the needle. In the early morning the north end of the needle moves slowly to the east reaching its extreme eastern position about 8 A. M. Then it moves to the west and reaches its extreme western position at about 1:30 P. M. The angle between these extreme positions varies from 5′ to 15′ of arc, depending on the season of the year. The needle has its mean position at about 10:30 A. M., and 8 P. M. The daily variation is so small that it is usually disregarded in compass surveying, but in determining the declination of the needle at any place it should be accounted for.

The compass needle is disturbed by the near presence of iron or steel or magnetic iron ores. The steel bows of eye-glasses, an adjusting pin, a knife, a watch, or a steel button will affect the needle if brought within a few inches of it. If the compass be set up near a railroad track, near iron pipes in the ground, near electric wires carrying heavy currents, near any iron or steel structure, or near a deposit of magnetic ore, its readings will be erroneous.

A disturbance of the needle due to any of the foregoing causes is called "Local Attraction."

THE COMPASS.

The compass is one of the most simple and convenient of instruments for measuring horizontal angles. Its principal advantage over other angle measuring instruments lies in the fact that it is always oriented. Its principal defect is that the orientation is inaccurate and varying, so that generally certain allowances or corrections must be made.

Referring to the parts which have been mentioned as essential in angle measuring instruments, there are found in the compass:

1. The pointers or sights which fix the particular line of the instrument that is brought into coincidence with the line of sight in the required direction.

2. A graduated circle upon which the direction of the sighting line is given in degrees of arc from the standard direction which is indicated by—

3. The magnetized needle which is balanced on a pivot at the center of the graduated circle, and which always lies in the magnetic meridian.

4. Various means are provided for making the graduated circle horizontal. Some compasses have leveling screws and plate levels like the transit,

others have a ball and socket joint and plate levels, and others are simply held horizontally in the hand.

5. The several parts of the compass are assembled in a case or box which is held in the hand if it is a pocket, or box, or prismatic compass, and is mounted on a tripod or Jacob's Staff if it is a surveyor's compass.

There are two general classes of compasses known *as needle compasses and card compasses.*

In the needle compass the graduated circle is attached to the box or plate and turns with the sights or pointers. In this case the index is the *north end* of the needle which remains fixed while the graduations of the circle pass by it.

In the card compass the graduated circle is marked on a circular card or on a light ring of metal which is attached to the needle, and which therefore remains fixed while the index which is marked on the box turns with the sights or pointers.

These two methods of reading angles may be compared with the two methods applied in reading the vertical and horizontal circles respectively of the transit. The vertical circle of the transit turns with the telescope while its index and vernier remain fixed. So, in the needle compass, the circle turns with the sights while the index or needle point remains fixed. The horizontal circle of the transit remains fixed while the index and vernier on the upper plate turn with the telescope. So, in the card compass, the graduated circle remains fixed with the needle, while the index on the box or case turns with the sights.

From these two methods of reading angles there result two corresponding methods of graduating the circular arc. All compasses should read zero (or 360°) when the sights point north, 90° when the sights point east, 180° when the sights point south, and 270° when the sights point west. In the needle compass remember that the index or north end of the needle remains fixed while the graduations of the circle pass by it. Then in turning from north, zero, to east 90°, the numbers of the graduation must run toward the left, contra-clockwise, and so on all around the circle; that is, the graduations must be numbered in a direction (left) opposite to the direction in which the angle is turned or measured (right).

In the card compass the circle, attached to the needle, remains fixed while the index with the sights or pointers turns past the graduations. Therefore in turning from the north, zero, to east 90°, the numbers of the graduations must run to the right or in clockwise direction. That is, the graduations must be numbered in the same direction (right) as that toward which the angle is turned or measured (right).

Unfortunately, compasses are not all graduated in the manner indicated above. Some are numbered from zero at the north and zero at the south, both ways to 90° at the east and 90° at the west. The same numbers are therefore found in each of the four quadrants, and in reading a bearing it is necessary to add the letters NE, SE, SW or NW, in order to designate the quadrant; thus, N 25° E, S 47° 30′ E, etc.

Others are graduated from zero at north, east, south and west to 45° at northeast, southeast, southwest and northwest, and in this case the letters indicating the octant must be added thus W 38° N, S 14° E, E 44° N, etc.

Some are graduated from zero at the north in both directions to 180° at the south, and the letter E or W must be added to indicate the semi-circle in which the angle is read.

There will also be found compasses numbered to read zero for a north pointing, 90° for a west pointing, 180° for a south pointing and 270° for an east pointing.

In the mariner's compass the divisions are not degrees, but *points*, arranged as shown in Plate A, Fig. 1.

A "point" is one-eighth of a quadrant, and is equal to 11¼ degrees. Smaller divisions are expressed as half or quarter points, thus, E ¼ N, read "east quarter north," or SW ½ S, read "southwest half south." This method of marking compasses is gradually giving way on modern vessels to the method of marking in degrees numbered to read zero when pointing north, 90° east, 180° south, 270° west, so that a bearing or course may be expressed by a number only and not by a complicated system of letters and fractions.

When a choice can be made, a compass numbered continuously from zero to 360 degrees, and reading N 0°, E 90°, S 180°, W 270°, should be selected. The surveyor, may, however, find that the only compass available is marked by any one of the systems mentioned, and should be able to use it and to record and plot its bearings.

A needle compass should have its graduated circle raised to the plane in which the needle swings, and the point of the needle should almost touch the inside of the circle. In some the circle is placed in the bottom of the box and the needle swings above it. These should be discarded when the other kind can be obtained.

The *Surveyor's Compass* has two sight leaves mounted at the ends of a plate. The compass is mounted in a circular box at the center of the plate. It has a raised circle, which is usually numbered as follows: The two zero marks are placed in a line with the sights and the numbers run both ways from each zero to 90° at the right and left. A *fleur-de lis* is engraved in the bottom of the box to indicate the forward or object end of the compass. The other is the eye end, and is indicated by S. The letter E is near the left hand 90° mark and the letter W near the right hand 90° mark.

When the "fleur-de-lis," or N zero, is at the north end of the needle the sights point north. Turning the sights 90° to the right brings the E 90° to the north end of the needle, and the sights point east. Another 90° turn brings the S zero to the north end of the needle, and the reading is south. Ninety degrees more brings the W 90° to the north end of the needle, and the pointing is west. Intermediate pointings are expressed thus, N 23° E, S 80° 30′ E, S 50° W, N 44° 15′ W, as previously explained. The smallest divisions of the circle are half degrees. With a reading glass the angle can be estimated on a large compass to the nearest 5 minutes. To read the bearing, note the degree mark that the north end of the needle has passed and add the fractional part. Record it by writing the letters of the quadrant with the number of degrees between them.

If the circle is numbered continuously from zero to 360°, no letters are needed, and this method of marking the circle is therefore to be preferred. In no system of measurement is it desirable to have more than one origin.

The graduated circle may, by means of a rack and pinion movement, be turned on the plate about its own axis. This motion of the circle makes it possible to use some direction other than the magnetic north as the standard direction from which to measure angles. Compare this arrangement of the compass with the similar feature of the transit. In the transit the horizontal circle may be turned at will and oriented with its zero line in any desired position. It may be set in the true meridian and give true asimuths, or it may be set in the magnetic meridian and give magnetic azimuths. So with the surveyor's compass, may the circle be turned and set to read zero when the sights lie in the magnetic meridian and thus give magnetic bearings, or to read zero when the sights lie in the true meridian and thus give true bearings. The difference between the two instruments is that in the transit the circle when oriented remains fixed, while the index and vernier turn with the telescope; in the surveyor's compass the index or north end of the needle remains fixed, while the circle when set by its vernier, turns with the sights. To set the circle of the compass in the desired position, there is provided an auxiliary circular arc with a vernier reading to minutes. Since the arc is generally used to set off the declination of the needle so that it shall read true bearings, it is called the *declination arc*, and its range and the corresponding motion of the circle are limited to include only the extreme declinations that may obtain in the localities where the compass will be used.

Incidentally the declination arc and its vernier may be used to read the fractional part or minutes of any bearing. Having pointed the sights in the desired direction, count the degrees to the mark that has been passed by the needle and read the declination vernier. Then turn the circle back by its rack and pinion movement till the needle is exactly opposite the degree mark, and read the vernier again. The difference of the two vernier readings will be the number of minutes to be added to the degrees. This refinement is not justified except when it is desired to determine accurately the angle between two lines by reading the bearing of each. In this case, local attraction and the variation of the needle do not affect the result. The vernier should also be used when an accurate determination of the declination of the needle is made.

The plate of the surveyor's compass is fastened centrally and perpendicularly to a socket which fits and turns on a spindle, the axis of which should be vertical. The spindle is mounted on the tripod or on a Jacob's staff by means of a leveling head or a ball and socket joint. Two level tubes are placed at right angles to each other on the plate to assure the verticality of the vertical axis. A clamp and tangent screw are provided to control the motion of revolution of the plate. A needle stop actuated by a milled headed screw lifts the needle from its pivot and holds it against the glass cover. The needle should be lifted from the pivot whenever the compass is not in use, or when it is carried.

For reading angles of elevation there is provided a peep sight near the bottom of the rear sight leaf and a tangent scale marked in degrees and half degrees on the front sight leaf reading upward. For reading angles of depression there is provided a peep sight near the top of the rear sight leaf and a tangent scale marked in degrees and half degrees on the front sight leaf reading downward. Sight through the peep hole at the distant point and note where the line of sight cuts the scale; the reading of the scale is the angle of elevation or depression as the case may be.

ADJUSTMENTS OF THE SURVEYOR'S COMPASS.

1. To make the plate levels perpendicular to the vertical axis. This adjustment is the same as the first adjustment of the transit, except that the compass has no telescope with attached level tube, and therefore the plate levels must themselves be used to make the axis vertical. Set up the compass and if it has leveling screws turn the plate till the plate level tubes are respectively parallel to the lines joining opposite screws. Bring the bubbles to the center of the tubes. Turn the plate 180°. If the bubbles remain at the center of the tubes the levels are in adjustment and the axis is vertical If they do not remain at the center, turn the leveling screws, or adjust the socket joint, so that the bubbles will move back over one-half of the displacement. This will make the axis of the compass vertical. Adjust the levels by means of their adjusting screws so that the bubbles shall return to the middle of the tubes. This makes the axis of the level tubes perpendicular to the vertical axis. Repeat the test to check the accuracy of the adjustment, and make further corrections if necessary. When the compass is again set up and lev-

eled so that the bubbles are at the middle of the tubes, the axis of the compass will be vertical.

2. To make the sight slits vertical. Having leveled the compass, sight through the slits at a plumb line. If either slit appears to be inclined to the plumb line the sight leaf is not vertical. The error may be corrected by filing the base of the sight leaf, or by gluing one or more pieces of paper under the side toward which the leaf inclines.

3. To straighten the needle. The ends of the needle and the pivot should lie in the same straight line. The following test and adjustment are independent of the position of the pivot, and are applicable whether the pivot is or is not at the center of the graduated circle. Read both ends of the needle. Revolve the compass so that the point on the circle that was at the north end shall come to the south end of the needle, and read the north end again. If its reading is the same as the original reading of the south end the needle is straight. If these two readings are not the same, bend the needle in the hands so that when it is replaced on the pivot, one-half of the apparent error shall have been corrected. The needle will then be straight. Repeat the test to check the result.

The test for this adjustment might be made by turning the needle by hand so that the south end would come around to the point previously occupied by the north end and then comparing the new reading of the north end with the original reading of the south end. The figure (Plate A, Fig. 2) illustrates the two positions of the needle and shows that whether the pivot be at the center of the circle or not, the test will make apparent the double error due to a bent needle, and hence will indicate the correction to be applied. C is the center of the circle and O is the pivot. N O S is the first position of the needle and N′O S′ is its reversed position, the points S′ and N coinciding. If the points S and N′ also coincide the needle is straight. If not, bend the needle till it can be made to occupy the position B O A, A being the midway between S and N.

The hand is not steady enough to turn the needle and hold it accurately in its reversed position, but exactly the same result is accomplished by turning the compass box as first explained. It makes no difference in this test whether the needle be turned on its pivot through the required angle, or the compass box be turned through the same angle while the needle remains at rest. It is easier in practice to turn the compass box, but the explanation is simplified by assuming that it is the needle that has been turned.

4. To straighten the pivot so that its point shall be at the center of the circle. After straightening the needle it may still be found that the readings of the opposite ends of the needle do not differ by 180°. This will be due to an eccentric position of the pivot point, and can only be corrected by bending the pivot so that its point shall be at the center of the circle.

When the compass is turned on the vertical axis, if the pivot point is eccentric this point will describe a small circle about the true center and will carry the needle with it first to one side and then to the other side of the true center, causing a varying difference of end readings. (Plate A, Fig. 3.) The discrepancy is zero when the line joining the true center and the pivot is parallel to the needle, and the discrepancy is a maximum when the line joining the true center and the pivot is perpendicular to the needle. To correct the error turn the compass to the position that gives the maximum discrepancy of end readings and then bend the pivot in a direction at right angles to the needle till the discrepancy disappears. The pivot point will then be at the center of the circle.

If there is a discrepancy of end readings of the needle that cannot be corrected by straightening the needle and the pivot, it is due to inaccuracies of graduation and cannot be corrected.

5. To adjust the vernier of the declination arc.

When the vernier of the declination arc reads zero the $0° — 180°$ line (or $0° — 0°$ line) of the compass circle should lie in the plane of the sight slits.

Stretch a fine thread from the bottom of one slit to the bottom of the other slit and see that it bisects the openings. Sight through the top of either slit downward toward the thread and turn the compass circle by its rack and pinion till the zero line coincides with the thread. If the vernier now reads zero it is correctly adjusted. If not, adjust the vernier without turning the circle, so that it shall read zero. If the vernier is not adjustable, its reading must be treated as an index error, to be applied in setting off declinations.

6. To remagnetize the needle.

A sluggish needle is a fatal defect in a compass and no reliance can be placed on its readings. If a compass be set away, when not in use, with its needle stopped in the reverse position with respect to the magnetic meridian, the needle will in time lose part of its magnetism. When again released the needle will move sluggishly and will not settle exactly in the magnetic meridian, being checked by the slight friction of the pivot. When a compass is put away, therefore, the needle should be allowed to settle in the magnetic meridian and then be lifted by the stop. In this position the earth's magnetism will tend to maintain that of the needle.

If the needle has become sluggish through loss of magnetism it must be remagnetized. This is best accomplished by placing the needle, with proper attention to polarity, in the field of a strong electro-magnet. A few minutes will serve to revive its magnetism.

If an electro-magnet is not available a permanent bar magnet may be used as follows: Stroke each half of the needle from the pivot toward the end with that end of the bar magnet that attracts the part stroked. In other words, stroke the north end of the needle with the south seeking end of the magnet and vice versa.

7. A sluggish needle may be due in part to a blunted pivot. When this is the case, unscrew the pivot and sharpen it on an oil stone, taking care to turn it constantly and preserve its conical form.

8. To test the scale of vertical angles.

A zero reading on this scale should indicate a horizontal line. Level the compass carefully and sight through the peep hole and the zero mark of the scale at a rod held vertically, and three or four feet from the compass. Mark the point pierced by the line of sight. Reverse the compass 180° and sight from the zero mark through the peep hole at the rod and mark the point now pierced by the line of sight. If the two points coincide in height, the zero line is horizontal. If the second point is below the first, the last line of sight inclined downward and the zero mark was too high. If the second point is above the first the zero mark was too low. The error may be corrected by glueing slips of paper under the lower sight leaf and thus correcting one half of the apparent error. Whenever glue is used a coat of varnish should be applied to protect the glue from dampness. Another way to make this correction is to bend the plate slowly and carefully in a vice, but this should not be attempted by a novice.

The compass attached to a transit is similar to the surveyor's compass, the telescope taking the place of the sights.

To make the fifth adjustment (vernier of declination arc) of the transit compass, proceed as follows: Set the declination vernier at zero. Lay a straight-edged ruler across the compass box and adjust it so that its edge shall coincide with the zero line of the compass graduations. Clamp the plates. Sight along the straight edge and set a mark in the line of sight so determined at a distance 100 feet or more. Sight through the telescope at the same mark. If the cross wires bisect the mark the adjustment is correct. If not, turn the telescope and bisect the mark. Turn the straight edge and again bisect the mark with it. Turn the compass circle by the rack and pinion movement and bring the zero line to coincide with the straight edge. Adjust the vernier so that it shall read zero. If not adjustable the error must be treated as an index error.

TO SET OFF THE DECLINATION.

Set up the compass and bring the needle and the declination vernier to a zero reading. If the declination be west D degrees, turn D degrees to the east. If the declination be east D degrees, turn D degrees to the west. The sights or telescope will now be pointing in the true meridian. Clamp the plates and with the rack movement turn the compass circle till the needle again reads zero, setting the declination off accurately by means of the declination vernier.

When the declination has been set off the compass will give true bearings.

THE PRISMATIC COMPASS.

The prismatic compass is a card compass with front and rear sights, the rear sight having an attached magnifying prism which reflects an image of the graduated edge of the card into the eye when a sight at the distant point is taken.

With any other form of the compass, the sighting line must first be pointed at the distant object and then the compass reading be made, but with the prismatic compass the graduated circle is read simultaneously with the pointing of the sights.

The index of this compass is the rear sight, and since it is desired that the compass read zero when pointing north, 90° when pointing east, etc., the zero should be placed at the south of the card, 90° at the west of the card, 180° at the north, and 270° at the east.

The mirror in the prism reverses the image of the graduation marks, and the numbers on the card must therefore be printed in reversed form like the face of the printer's type, in order that the mirrored image may be direct.

The prismatic compass is the most accurate of the hand compasses and is a valuable instrument for rough surveying or careful sketching.

BOX OR MILITARY COMPASS.

This is a simple needle compass mounted in a rectangular block of wood with a hinged cover. In some, the edge of the cover when raised to a vertical position is the sighting line, and in this case, holding the cover toward the right, the zero should be at the forward side, the 90° mark at the left, the 180° mark at the near side and the 270° mark at the right.

In another form a white line is marked on the inside of the lid. When the lid is wide open this line is in the prolongation of the 0°—180° line of the compass. To point the compass, open the cover about 120° and turn it to the front. Hold the compass in a level position and a little below the eye. The white line will be seen reflected in the glass top of the compass. Make this reflected line coincide with the 0°—180° line of the compass and while maintaining this coincidence sight over the top of the white line at the distant object. When the needle comes to rest, read its north end.

Many forms of pocket compasses are manufactured, but they are generally unsuitable for surveying or sketching. For these purposes a needle compass should have the following features:

1. A good sighting line.
2. A light sensitive needle with a stop.
3. A raised circle lying in the same plane with the ends of the needle.

4. Plainly marked graduations dividing the circle into single degrees and numbered continuously from zero to 360° in counter-clockwise direction. The 0°—180° line should be parallel to the sighting line, with the zero at the forward side.

5. A strong, water-tight case.

For the card compass the requirements are the same except that the graduated circle is marked on the card, the numbering should be in clockwise direction, and the index should be so placed as to read zero when the sights point north.

To plot the bearings given by a compass.

It has been explained that compasses are graduated and marked in several different ways, and that the same direction with reference to the meridian may be expressed by a number of different compass readings, depending on the kind of compass and the manner of its marking. In order, therefore, to plot the bearings given by any particular compass the protractor should be marked or numbered to correspond with the *readings* of the compass. This does not mean that the protractor should be marked like the circle of the compass used, but rather that it should be marked to give the same *readings* as the compass.

For example, a needle compass, like the box compass, that reads zero when pointing north, 90° when pointing east, 180° when pointing south and 270° when pointing west, is graduated to the left or counter-clockwise and has the zero mark at the forward side, the 90° at the left, 180° at the near side and 270° at the right, whereas a prismatic compass that gives the *same readings* would be graduated to the right or clockwise, and would have the zero at the south point.

of the card, 90° at the west point, 180° at the north, and 270° at the east.

For both of the compasses just described the markings of the protractor should be the same and should be numbered in clockwise direction from zero to 360°, and the zero mark should be placed toward the north end of the meridian when the center mark is at the station point.

The rule to be followed for determining the proper marking of a protractor to correspond with any compass is as follows:

Point the sighting line of the compass to the north and read the compass. Mark this reading on the protractor at the north end of the diameter that is to be placed in coincidence with the meridian line on the plot. Next point the compass to the east and read it. Mark this reading on the protractor at a point 90° to the right of the north reading. Then point the compass to the south and read it. Mark this reading at the south end of the meridian diameter of the protractor. Lastly, point the compass to the west and read it. Mark this reading on the protractor at a point 90° to the left of the meridian diameter. Mark also the intermediate points of the protractor at 10° (or 5°) intervals to correspond with intermediate compass readings. The protractor will then plot, without reduction, the recorded readings of the compass.

If the protractor has been marked by the maker with numbers that do not correspond with the compass used, it is only necessary to disregard those numbers and mark the proper numbers with pencil or ink. Conversely, if the compass is not marked in the desired manner, new numbers may be added and the old ones disregarded.

A *plotting diagram* for any compass consists of two lines crossing at right angles, and marked at their ends with the N, S, E and W *readings* of the compass. It indicates the proper markings of the protractor to be used with that compass. The figures on Plate A, Fig. 4, give the compass graduations and corresponding plotting diagrams for four compasses, the first two being box or needle compasses, and the other two being prismatic or card compasses. For any other compass the plotting diagram would be constructed in a similar manner.

In military topographic sketching it may be necessary for compass notes taken by one person to be plotted by another. Even when the sketcher plots his own notes he may be working with a compass furnished him for the particular occasion and differing in its marking from the one he is accustomed to using. To remove all chance of error a plotting diagram of the compass used should be placed at the beginning of every set of compass notes.

COMPASS SURVEY.

PARTY	EQUIPMENT
Surveyor	Compass, note book, etc.
Front chainman	Crayon and marking pins
Rear chainman	Chain and note book
Rod and axeman	Rod and axe

The principles and methods involved in surveying with compass and chain are the same as those of a transit survey except that distances cannot be measured with the stadia, and must be chained.

The surveyor directs the work and uses the compass. He sets up the compass at the selected initial station and sees that the declination vernier reads zero, if magnetic bearings are desired, or that it reads the proper declination, if true bearings are desired. He reads bearings and vertical angles to selected critical points and directs the chainmen to measure the horizontal distance from his station to these points. The last observation at station zero is made to determine station 1, which is so selected as to give a good view of surrounding critical points and to advance the survey in the desired direction. He records bearings, vertical angles, distances and points observed, on the left hand page of the note book, and on the right hand page makes a free hand sketch of the ground covered, showing all points observed and the objects and features so determined.

FORM OF RECORD.

(Title page, description of instrument, names of party, etc., same as for transit survey:)

Bearing.	Vert. Ang.	Hor. Dist.	Diff. Elev.	Elev.	Point Observed.
At	station	0.	Ele	v. 568.	
N. 56° E.	—4°	332	—23.2	544.8	Fence cor.
et S. 5½ E.	c. +3½	428	+26.1	etc. 541.9	Sta. 1
At N. 5¼ W.	station —3¾	1. 428	Ele —27.8	v. 541.0 540.2	B. S. Sta. 0

If only a boundary or a line is to be run without reference to topography, the columns, Vert. Ang., Diff. Elev., and Elev. are omitted.

Having taken all necessary observations at station zero the surveyor stops the needle and carries the compass to station 1. The first sight from station 1 is a backsight at station zero, taken to verify the forward readings from that station of bearing and Vert. Ang.

The operations already described are repeated at successive stations to the end of the traverse, which should, if possible, close on the initial station or on a known point.

The chainmen measure all required distances as directed by the surveyor. The operation of chaining will be described under the head of "Measurements of Distance."

The rodman is provided with a staff on which is a mark whose height above the ground is equal to the average of the heights of the two peep holes on the rear sight leaf of the compass. The rodman holds this staff at the "point observed and the surveyor sights at the mark when reading vertical angles. Great accuracy in determining elevations cannot be expected by this crude method of measuring vertical angles, but with care fair results can be obtained in a small survey.

Since horizontal distances are measured, the difference of elevation is obtained by multiplying the horizontal distance by the tangent of the vertical angle. The elevation of a "point observed" is found by adding algebraically the "difference of elevation" to the "elevation" of the station occupied.

After the notes are reduced, the adjustment of the traverse, the plotting of the traverse, the plotting of the side shots, and the drawing of the map, are the same as the like operations in a transit survey.

When the foresight and backsight bearings between two stations do not agree it is probable that there is "local attraction" at one of the stations. Select a third station at as great a distance as is convenient from the other two and read foresights and backsights on each side of the triangle. The station at which all foresights and backsights disagree is the one probably affected by local attraction. If the foresight and backsight agree between the two other stations they are probably not affected. A station affected by local attraction should be abandoned if one not so affected can be selected. In regions containing deposits of magnetic ores, local attraction is so general that compass bearings cannot be relied on. In this case the compass may be oriented at each station in a manner analogous to that applied in orienting the transit; thus, backsight at the previous station and turn the circle with the pinion movement so that the backsight shall agree with the foresight.

PLATE A

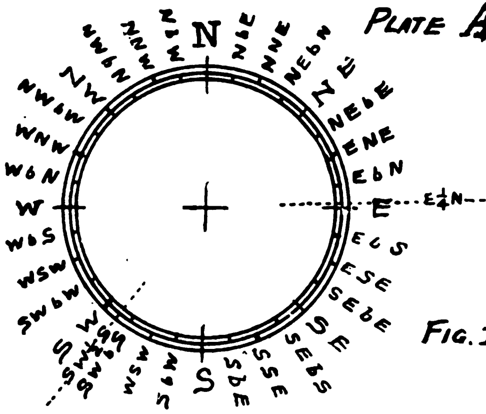


FIG. 1

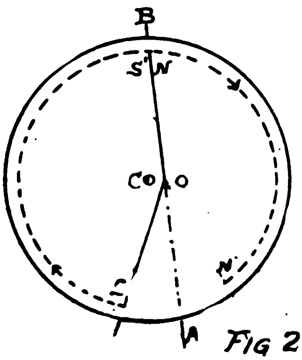


FIG 2

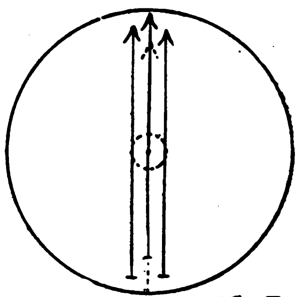
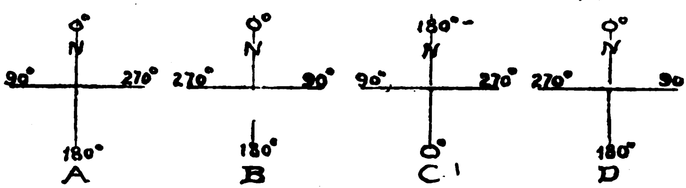


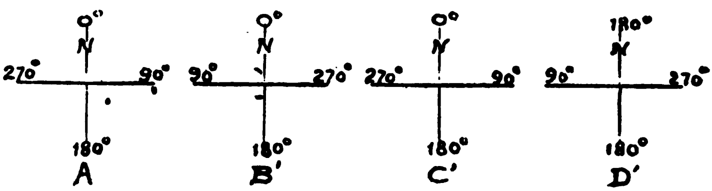
FIG 3

COMPASS GRADUATION – FIG. 4

BOX COMPASS　　PRISMATIC COMPASS



PLOTTING DIAGRAM



CHAPTER VII.

TO DETERMINE TRUE AZIMUTH.

The *true azimuth* of a line is the angle which it makes with the true meridian at the point considered, measuring from the north point toward the right.

The pole star (Polaris) appears to describe a small circular orbit about the true north pole of the celestial sphere. At the most easterly point of its orbit it is at *eastern elongation:* at the most westerly point of its orbit it is at *western elongation.* At its elongations the star is moving apparently in a vertical direction, upward at eastern, and downward at western elongation, and for about ten minutes before and after elongation its deviation from the vertical is not appreciable. Therefore, at these times it is fixed in azimuth, and its azimuth may be found as follows:

$$\text{sine of azimuth} = \frac{\text{sine of polar distance}}{\text{cosine of latitude}}$$

The polar distance is the angular distance of the star from the pole and is given in the following table (approximate):

POLE DISTANCE OF POLARIS.

1906	1909	1912	1915	1918	1921	1924	1927	1930
1° 12′	1° 11′	1° 10′	1° 9′	1° 8′	1° 7′	1° 6′	1° 5′	1° 4′

For intermediate years interpolate in the above table.

Find the pole distance for the year and divide its sine by the cosine of the latitude; the result is the sine of the azimuth of Polaris at elongation, and the azimuth may be found in a table of sines.

Thus at Fort Leavenworth in 1908,

$$\text{Pole distance} = 1^\circ\ 11.'3$$
$$\text{Latitude} = 39^\circ\ 20'$$

$$\sin \text{azimuth} = \frac{\sin 1^\circ\ 11.'3}{\cos\ 39^\circ\ 20'} = \frac{.02085}{.77347} = .02696$$

$$\text{Azimuth} = 1^\circ\ 32.'6$$

The observations on Polaris for azimuth must be made at the time of elongation. To find the time of elongation proceed as follows: The times in the table are for the nights *following* the given dates

MEAN LOCAL TIME OF ELONGATION OF POLARIS.
YEAR 1901. LATITUDE 40° N.

Date.	Elongation.	Time.		Culmination.	Time.	
		hrs.	min.		hrs.	min.
Jan. 1	W	12	34.8 a. m.	U	6	39.8 p. m.
Feb. 1	W	10	32.4 p. m.	L	4	35.5 a. m.
Mar. 1	W	8	41.9 p. m.	L	2	45.0 a. m.
Apr. 1	W	6	39.8 p. m.	L	12	42.9 a. m.
May 1	E	4	48.1 a. m.	L	10	45.1 p. m.
June 1	E	2	46.7 a. m.	L	8	43.6 p. m.
July 1	E	12	49.2 a. m.	L	6	46.1 p. m.
Aug. 1	E	10	47.8 p. m.	U	4	42.7 a. m.
Sept. 1	E	8	46.4 p. m.	U	2	41.3 a. m.
Oct. 1	E	6	48.7 p. m.	U	12	33.6 p. m.
Nov. 1	W	4	36.7 a. m.	U	10	41.9 p. m.
Dec. 1	W	2	38.6 a. m.	U	8	43.6 p. m.

For *other days* in any month subtract from the tabular time 3.94 min. for each day after the first.

For *other years* after 1901, add 0.4 min. for each year and also,

Add 1 min. in the second year after leap year.

Add 2 min. in the third year after leap year.

Add 3 min. in leap year before March 1.

Subtract 1 min. in leap year after March 1.

Add nothing in the first year after leap year except the periodic change of 0.4 min. per annum after 1901.

For *other latitudes* between 30° and 50° N:

For each degree south of 40°.

Add to time of W. elongation, 0.14 min.

Subtract from time of E. elongation, 0.14 min.

For each degree north of 40°.

Subtract from time of W. elongation, 0.18 min.

Add to time of E. elongation 0.18 min.

Example: Find the time of eastern elongation on September 15, 1907.

		H. M.	
For Sept. 1, 1901,		8:46.4	P. M.
For Sept. 15, 14 days × 3.94 =	—	:55.16	
		7:51.24	
For 1907, 6 yrs × 0.4 =	+	2.4	
		7:53.64	
For third year after leap year	+	2.0	
		7:55.64	
For Lat. 39° 20′ N. ⅔ deg. S. of 40°, ⅔ × 0.14 =	—	0.9	
Mean local time E. elongation =		7:54.74	P. M.
At Fort Leavenworth, standard time is faster than local time by		:19.7	
Standard time of E. longation at Fort Leavenworth.	=	8:14.44	P. M.

Since observations on Polaris must be made at night the cross wires of the telescope must be illuminated, and the line whose azimuth is desired must be given by an artificial ray of light from a lantern or lamp placed on the line.

To illuminate the cross wires an assistant throws the light from a bull's eye lantern obliquely into the objective, or better, a reflector made of tin is attached to the telescope at angle of about 45°. The light from a bull's-eye lantern held at one side is thus reflected into the tube of the telescope. The reflector must cover only a small portion of the objective and is best arranged in the form of an elliptical ring covering only the edge of the objective and giving an unobstructed view through the central opening. An attached strip of tin bent around the end of the tube holds the reflector in place.

To illuminate a mark or target on the line whose azimuth is desired, place a lantern or lamp in a box which has a vertical slit one-half inch wide in the side toward the transit. The slit should be covered with thin white cloth or paper. Or better, screen one side of the box with tracing paper or tracing linen on which is drawn a heavy black vertical line. The vertical slit or the vertical line must be centered exactly on line at a distance of 500 yards or more from the transit station.

An observation for azimuth consists simply of reading the horizontal angle between Polaris at elongation and the lighted terrestrial mark. Since the star and the mark are at very different elevations, faulty adjustment of line of collimation and horizontal axis will cause serious error. Therefore, several readings are taken with telescope alternately direct and reversed.

Light up the mark and set up the transit, carefully leveled, so that everything is ready 15 minutes before time of elongation. Then follow this programme (Johnson's Surveying):

Instrument.	Time of Observation.	Reading on:
Direct	10 minutes before elongation	Mark
Reversed	7 minutes before elongation	Mark
Reversed	4 minutes before elongation	Star
Reversed	2 minutes before elongation	Star
Direct	2 minutes after elongation	Star
Direct	4 minu-es after elongation	Star
Direct	7 minutes after elongation	Mark
Reversed	10 minutes after elongation	Mark

An assistant illuminates the cross wires and records the readings of both verniers given him by the observer.

Take the means of all vernier A readings on star and on mark respectively and find their difference.

Take the means of all vernier B readings on star and on mark and find their difference.

Take the mean of the two differences as the angle between the star and mark.

To this angle, add the azimuth of Polaris if at E. elongation or subtract the azimuth of Polaris if at W. elongation. The result will be the true azimuth of the line from transit station to illuminated mark.

TO FIND THE DECLINATION OF THE COMPASS NEEDLE.

The difference between the true azimuth and the magnetic bearing of a line at any point is the declination of the needle at that point. Therefore, point the telescope of the transit along a line whose true azimuth is known, and read the north end of the needle.

Let positive angles be figured to the right or in clockwise direction, azimuth from true north and magnetic bearing from magnetic north, and let

$Z =$ azimuth
$M =$ magnetic bearing
$D =$ declination of needle.

Then $Z - M = D$. (Plate C, Fig. 1.)

A positive result denotes that the declination is *east*, or that the needle points D degrees in clockwise direction from true north.

A negative result denotes that the declination is *west*, or that the needle points D degrees in contra-clockwise direction from true north.

If the result D be greater than 180° subtract it numerically from 360° and give the result a sign opposite to that of D. Thus, if $D = +350^\circ$, take $360^\circ - 350^\circ = 10^\circ$, and -10° is the declination, denoting that the magnetic north lies 10° west of the true north.

In reading the north end of the needle, only degrees or half degrees can be read by the divisions of the graduated circle. When the needle lies between two divisions, count the divisions to include the one preceding the needle and read the plate vernier. With the tangent screw turn the plate back so that the needle points exactly at the preceding division and read the vernier. The difference of the two vernier readings gives the number of minutes to be added to the needle reading.

The needle has a small daily variation, the north end swinging slowly to the west from 8:00 A. M. to 1:30 P. M., and to the east from 1:30 P. M. to 8 A. M. It has its mean or true declination at 10:30 A. M. and 8 P. M.

Therefore an observation for declination of the needle should be taken at 10:30 A. M., local time.

Knowing true azimuth and declination, to find magnetic bearing.

$$Z - D = M.$$

The result to be expressed algebraically. If M is positive the bearing is to be figured in clockwise direction; if negative in contra-clockwise direction from the magnetic north.

Knowing magnetic bearing and the declination, to find true azimuth.

$$M + D = Z.$$

As before the result is to be expressed algebraically, with attention to signs. In either case if the result be greater than 360°, subtract 360° from it.

When a surveyor's compass is used, a sight cannot be taken direct at Polaris except in low latitudes. To establish a line of known azimuth proceed as follows: Find the azimuth of Polaris at elongation and time of elongation, as already explained. Suspend a plumb bob by a fine thread from a high point, as from an upper window or the limb of a tree, and let the plumb bob dip in a bucket of water to check its oscillations. About 20 minutes before elongation drive a heavy stake in the ground south of the plumb line and in line with plumb line and star. The stake should be high enough to be sighted over conveniently from a sitting or prone position on the ground. At time of elongation sight over the stake at plumb line and star and set a needle or pin vertically in the top of the stake and exactly in line with plumb line and star. Leave the plumb line and needle undisturbed, and on the following morning prolong the line to the north by setting a distant mark on line. At 10:30 A. M., local time, set up the compass with its center over the needle and point at the distant mark. Turn the circle with the declination movement so that the reading of the needle shall be equal to the known azimuth of the line, using the declination vernier to set off the minutes. The compass is now oriented to read true bearings, and the reading of the declination index and vernier gives the declination of the needle.

To *mark a true meridian*, measure the horizontal distance from the needle or pin in the top of the compass station to the distant mark and multiply the distance by the tangent of the azimuth of the line. The result is the distance to be laid off from the distant mark at right angles to determine a point on the meridian through the compass station. Two permanent marks should be set on this line. At any subsequent time the compass may be set up over the south mark, and the declination of the needle determined by a pointing at the north mark.

If the latitude of the place is not known it may be readily determined by an observation on Polaris at culmination. In its apparent circular orbit, Polaris crosses the meridian twice, once above the true pole at upper culmination and once below the true pole at lower culmination.

Let $L =$ latitude of place.
$A =$ altitude of star.
$P =$ pole distance of star.
$R =$ refraction.

Then $L = A - R \pm P$,

the plus sign being used for lower culmination and the minus sign for upper culmination.

The time of culmination is taken from the table on page 188, all the reductions being applied excepting that "for other latitudes."

The pole distance, P, is found by interpolation in the table on page 187.

The refraction, R, may be taken from the following:

TABLE OF MEAN REFRACTION.

Altitude.	Refraction.	Altitude.	Refraction.
10°	5′ 19″	20°	2′ 39″
11	4 51	25	2 04
12	4 28	30	1 41
13	4 07	35	1 23
14	3 50	40	1 09
15	3 34	45	0 58
16	3 20	50	0 49
17	3 08	60	0 34
18	2 58	70	0 21
19	2 48	80	0 10

Set up the transit at the selected station and see that it is carefully adjusted, especially with reference to the plate levels. One pair of leveling screws should lie in the north and south line. A bull's-eye lantern is needed to illuminate the cross wires and to read the vernier.

About five or ten minutes before culmination read the altitude of the star. Reverse and plunge the telescope, and if the plate bubbles have moved relevel the instrument. Take two readings of the altitude of the star in this position. Reverse and plunge the telescope, relevel the instrument, and take one reading of altitude.

The reversals and releveling of the instrument have eliminated errors of collimation, of plate levels, and of index, and the mean of all the readings is the apparent altitude of the star. The correction for refraction gives the true altitude, and the correction for pole distance gives the altitude of the pole, which is equal to the latitude of the place.

If the vertical limb of the transit is an incomplete circle, the method just described will not apply, and an *artificial horizon* must be used. This is a small shallow pan of mercury placed on the ground in front of the instrument in such a position that the mercury image of the star can be seen with the telescope. Begin a few minutes before culmination and take one reading on the star, then two readings on the mercury image, then one reading on the star. The mean of all the readings will be the apparent altitude of the star and the corrections for refraction and pole distance will give the latitude.

The sextant and artificial horizon may be used to read the vertical angle between the star and its image, as will be explained under the head of "The Sextant."

An approximate determination of local time and of longitude may be made by taking the standard time of the sun's crossing the meridian and applying the "equation of time," which is the correction to be applied to apparent sun time to get mean or correct time. A good watch is required, and it should be compared with standard time at a railway station or telegraph office before and after the observation, so that time can be read to the nearest half second.

Having determined the true azimuth of a line, set up the transit at one station on the line and point accurately at the far station. The transit must be in careful adjustment, especially the plate levels, and one pair of leveling screws should lie in an east and west line.

While pointing on line, read both verniers and subtract from each reading the azimuth of the line. This will give the meridian reading of the verniers. Set the verniers at this meridian reading, and the telescope will lie in the meridian. A neutral glass must be used in front of the objective. This consists of a thick piece of red or green plate glass, ground to plane parallel faces and mounted in a ring that fits the end of the telescope. A neutral eyepiece would shield the eye but not protect the cross wires from the heat in the focus of the sun's rays. A prismatic eyepiece, turning the rays 90°, is necessary because when pointing at the sun the eye end of the telescope is close to the plate.

When the sun approaches the meridian, verify the readings of the verniers, and level carefully. Caution "Ready" to the assistant who holds the watch. Point towards the sun and when its western limb becomes tangent to the vertical wire, call "Tick."

Reverse and plunge the telescope, set the verniers again at the meridian readings, relevel the instrument, caution "Ready," and when the sun's eastern limb becomes tangent to the vertical wire, call "Tick" again.

The assistant takes the time to the nearest half second at each "tick" and records it. The mean of the two times is the watch time of apparent noon. Compare the watch again with standard time and find its error at the time of the observation. Applying this correction will give standard time of apparent noon.

Find in a "nautical almanac" for the year the "equation of time" for the day and hour of observation and apply it to standard time of apparent noon; the result will be standard time of mean noon, or difference of longitude in time, between the standard meridian and the place of observation.

Thus, if the standard 90th meridian time of mean noon be 12. h. 19 min. 42 s. P. M., the place of observation is 19.7 min. west of the 90th meridian, and since 1 min. of time is equal to 15′ of longitude, the longitude of the place is 94° 55′ 30″ W.

CHAPTER VIII.

THE SEXTANT.

(Plate B, Fig. 1.)

The Sextant is a hand instrument for measuring the angle between two rays of light from two distant points, one ray coming direct to the eye and the other ray reaching the eye by double reflection in two mirrors, one of which is movable. The angular motion of the movable mirror is indicated on a graduated arc so marked as to give the angle included between the lines of sight to the distant points. The angle is measured in the plane that includes the eye of the observer and the two distant points. The greatest angle that can be measured is about 120° to 140°.

The instrument consists of a frame at one side of which is the graduated arc. At the center of this arc is pivoted the movable mirror, called the index glass, which faces toward the front and is mounted on an axis perpendicular to the plane of the arc. To this glass is fastened an arm which carries the index and vernier. The motion of this index arm and its glass is controlled by a clamp and tangent screw.

Opposite the index glass and facing the observer is the fixed mirror called the horizon glass. Only the lower half of this glass is silvered, the upper half being clear.

Looking into the horizon glass the observer has a direct view through the clear portion at a selected distant point. In the silvered portion he sees an image of the index glass and of the objects that are mirrored in it. By turning the index glass the reflected image of a selected point may be brought into view and into coincidence with the first point seen direct. When this coincidence is secured, the reading of the index and vernier gives the angle between the lines of sight to the two selected points.

The ray of light from the first point D comes direct to the eye. The ray of light from the second point R, has been deviated by reflection in two mirrors, so that its final direction coincides with the direction of the first ray.

Let I represent the index glass;
H, the horizon glass;
V, the vernier on the index arm;
C, the graduated arc;
E, the eye of the observer, and let
m = the angle at M between the mirrors;
= the angle at N between the perpendiculars to the mirrors;
i = the angles of incidence and reflection at the index glass, I;
h = the angles of incidence and reflection at the horizon glass, H.

Then in the triangles I N H and I A H respectively:

$$m = i - h$$

$$\text{and } A = 2i - 2h$$

$$\text{hence } A = 2m \text{ or,}$$

the angle A between the rays of light from the two points R and D is twice the angle, m, between the mirrors.

When the two mirrors are parallel, that is, when the index glass, I, lies in the plane I K parallel to H M, then

$$m = 0, \text{ and } A = 0$$

or the two rays of light are parallel and the points R and D coincide. In other words, when the mirrors are parallel the same point will be seen, direct in the clear part, and reflected in the silvered part of the horizon glass. For this position of the index glass, therefore, the index should read zero on the graduated circle, at some point, as K.

When the index glass and arm are turned to some other position as I M, the arc K V = m, over which the vernier passes is one-half the angle A between the direct and reflected rays of light, and therefore every half degree on the arc must be counted and marked as a whole degree in order that the required angle, A, may be indicated. The arc of the instrument actually includes only 65 or 70 degrees, but it is marked to read 130 or 140 degrees. For larger angles, the index glass becomes so oblique to the direction of the rays of light, that a very small field of view is presented. The effective arc is about 60 degrees, hence the name "sextant." Some instruments of smaller range are called "octants."

With the naked eye, one of the distant objects is seen through the clear glass and the other in the silvered glass. They are therefore separated by the line at the edge of the silvered portion and cannot be superimposed. When a telescope is used, the upper part of its lens forms a complete image of one object, and the lower part of its lens forms a complete image of the other object. There is no dividing line, and the two images may be exactly superimposed. For this reason, and also as an aid to distinct vision, the telescope is generally used. It must be mounted in such a position that it will see, in the horizon glass, an image of the index glass, and its axis must be parallel with the plane of the instrument.

When the telescope is not used, its place should be taken by a peep sight in order that the eye may be placed at the proper point for viewing the reflected image and that the line of sight across the edge of the silvered part of the horizon glass may be parallel with the plane of the instrument.

In observing the sun, dark or colored glasses must be interposed to shield the eye. These "neutral glasses" are conveniently arranged to be turned into the path of either the direct ray or the reflected ray at will. A neutral eyepiece is also provided.

In the larger sextants the radius of the arc is about 7 inches, which is more than twice the radius of the best transits. The arc is often graduated to 10 minutes and the vernier reads to 10 seconds. The sextant is therefore an instrument of precision, and is one of the most accurate of angle measuring instruments.

The *pocket sextant* is a small instrument enclosed in a cylindrical metal case about 3 inches in diameter. In principle, its construction is the same as that of the larger sextants.

ADJUSTMENTS OF THE SEXTANT.

First Adjustment. To make the index glass perpendicular to the plane of the graduated arc.

Set the vernier at about the middle point of the arc. From the pivot side of the instrument sight across the top of the frame at the arc and obliquely into the index glass. The view of the arc will be interrupted by the glass, but in the glass will be seen a reflected image of the arc. This reflected image should appear to form a continuous circle with the portion of the arc seen direct. If it does not, adjust the mirror till the apparent continuity of the circle is secured. The mirror will then be perpendicular to the plane of the arc.

Second Adjustment. To make the horizon glass perpendicular to the plane of the circular arc.

Having made the first adjustment, sight through the telescope at a distinct, well defined, distant point, (a star is best) and bring its reflected image into coincidence with the direct image. If exact coincidence can be secured the horizon glass and the index glass are known to be parallel, and since the index glass has been made perpendicular to the plane of the arc, the horizon glass must also be perpendicular to that plane. If, however, the reflected image passes above or below the direct image, it is known that the horizon glass inclines, respectively, towards or from the observer, and it must be so adjusted that exact coincidence of the two images can be secured.

When a peep sight is used instead of a telescope the two images cannot be exactly superimposed because one is seen in the silvered part and the other in the clear part of the horizon glass. In this case the second adjustment may be made as follows: Sight at a well defined straight line such as the ridge of a house, the corner of a building or a distant horizon, holding the sextant in a plane that is perpendicular to the line sighted at. Bring the reflected image of the line into coincidence or prolongation with the direct image, then tilt the sextant at an angle of about 45 degrees with its first position. If the two images remain in coincidence the adjustment is correct. If not, adjust the horizon glass so that the coincidence shall not be disturbed when the sextant is tilted from its first position. This method is applicable when the telescope is used, but is not to be preferred.

Third Adjustment. To make the line of collimation of the telescope parallel to the plane of the circular arc. In the telescope of a sextant there are usually two parallel wires in one direction and two other wires crossing the first two at right angles. These wires form a small square at the center of the field of view, and the line of collimation is the straight line through the center of this square and the center of the object lens. By this arrangement, the wires do not obstruct the view at the point where the coincidence of images is to be made.

To test for the third adjustment place the sextant on a firm support, such as a table or a wall, and sight through the telescope at a rod or pole planted at a short distance and mark the point intersected. Then sight across the frame of the sextant in the plane of circular arc and mark the point on the rod cut by this plane. Move the rod to a greater distance and mark the two points determined by sighting at it in the same manner as before. If the two intercepts thus marked on the rod are equal, the line of collimation of the telescope is proved to be parallel to the plane of the circular arc. If not equal, the telescope must be adjusted so that the intercepts determined, as described, shall be equal for the different positions of the rod.

When a peep sight is used, its height above the plane of the instrument should be equal to the height of the edge of the silvered part of the horizon glass above the same plane.

Parallax. By examining the figure (Plate B, Fig. 1) it will be seen that the vertex A of the angle measured moves along the line E D as the angle changes, and that when the angle is small the point A may be some distance in rear of the observer. In using the sextant, therefore, the exact position of the vertex of the angle measured is never known, and the error due to this cause is called instrumental parallax. This error is usually very small, and may be neglected; but when the angle itself is small and the object seen by reflection is only a short distance from the observer, the error becomes appreciable. To eliminate this error, select a third point at an angle of about 60° from the others and measure the angles between this third point and each of the other two points. The difference of these two angles will be the required angle unaffected by parallax.

Index Error. Select a distant, well defined point (a star is best) and bring the reflected and direct images into exact coincidence. Read the vernier. If the reading is zero, the mirrors are correctly set and there is no index error. If the reading is not zero, its amount is the index error, positive if the index is to the right of the zero, or "off the limb," and negative if the index is to the left of the zero, or "on the limb."

In order to read the vernier when the index is "off the limb," the graduations of the limb are continued to the right beyond the zero, and this added arc is called the "arc of excess." On this arc of excess the vernier must be read backwards, that is, its divisions must be counted from left to right.

On some sextants, one of the mirrors is made adjustable by having a slight motion of rotation on its support. When this is the case the index error may be corrected as follows: Set the vernier at zero. Sight at a distant, well defined point, preferably a star, and with the adjusting screw that turns the adjustable mirror, bring the two images of the point into exact coincidence. When this is done the index error is zero.

To measure with a sextant the angle between the lines of sight to two distant points. Hold the sextant in the right hand with the left hand on the clamp screw. The grasp of the handle should be rather loose and flexible, not rigid. Hold the instrument in the plane of the two points and sight through the telescope at the left hand point, keeping its image in the center of the field of view. Sweep the index arm till the other point comes into view. Then clamp the arm and secure exact confidence of the two images by means of the tangent screw. Read the index and vernier. This reading, corrected for index error, if any, will be the required angle.

When the two points lie in the same vertical plane, hold the instrument in that plane and direct the telescope at the lower point. Bring the upper point into view by sweeping the index arm as before described.

The image of the object seen direct is brighter and more distinct than that seen in the mirrors because some light is lost by double reflection. It is sometimes better, therefore, to view the dimmer object direct and the brighter by reflection. To do this when the right-hand object is indistinct, the sextant must be held upside down in the left hand, with the right hand on the tangent screw.

But the brighter of the two objects is generally the nearer, and the error due to parallax will be smaller if the nearer object be viewed direct.

The two conditions affecting brightness of image and parallax respectively may therefore be conflicting, and the judgment of the observer must decide as to the more important of the two.

In some sextants the ring that holds the telescope is adjustable. By raising the telescope from the frame, the direct image becomes brighter, since it gets more light through the upper clear portion of the horizon glass. By lowering the telescope the reflected image becomes brighter, since it then gets more light from the lower silvered portion of the glass. The two images may thus be made equally distinct.

To measure altitudes, or angles of elevation above the horizon.

At sea, the altitude of the sun or star is referred to the sea horizon. Hold the sextant in a vertical plane and, sighting direct at the horizon line, bring the reflected image of the sun or star tangent thereto.

The correction to be applied to the observed angle to get correct altitude are $\pm$ index error; — dip of the horizon, — refraction, and, for the sun, + the sun's semi-diameter.

To measure altitudes on land an *artificial horizon* must be used. This ordinarily consists of a rectangular shallow pan containing mercury. To prevent disturbance by the wind or clouding by dust, a cover with inclined glass windows is used. The surface of the mercury forms a horizontal mirror in which the reflection of any point is seen at an angle of depression that is equal to the altitude of the point. The angle included between the point and its mercury image is therefore twice the altitude of the point and is called the "double altitude."

With the sextant, sight direct at the mercury image and bring the image from the mirrors into coincidence by sweeping the index arm. The observed angle, corrected for index error, is the observed double altitude. Half of this double altitude corrected for refraction is the altitude of the point.

When the sun is observed, its two images are not superimposed, but are made tangent to each other. The lowermost point of the sun's circle (called its lower limb) is mirrored in the mercury as the uppermost point of its reflection. By bringing these two points into contact in the sextant the altitude of sun's lower limb will be determined. This must be corrected by adding the sun's semi-diameter in order to get the altitude of the sun's center.

With an inverting telescope the uppermost point of the moving image is the sun's lower limb, and is the point that must be brought into contact with the other image. Tangency can be made at the upper limb if desired, but the same limb must be used in the same set of observations. To remove all doubt as to the identity of the two images, interpose neutral glasses of different colors in the paths of the two rays. Thus, if the mercury image be made red and the index-glass image be made green the two images need never be confused.

Sextant observations on sun and stars are made to determine latitude, longitude and time.

In measuring the double altitude of a terrestrial point, the sextant should be held as close as possible to the artificial horizon in order that the rays of light from the point to the sextant and to the artificial horizon respectively, may be nearly parallel. The nearer the point observed, the greater will be the divergence of these rays, and hence the greater will be the error in the measured altitude.

If the telescope of the sextant be replaced by a hand level, centered in the telescope ring, vertical angles referred to the horizontal may be measured without the artificial horizon. Hold the sextant in the vertical plane that contains the object. Sight through the hand level and bring the bubble to the middle of its run or to the point where it appears to be bisected by the cross wire. The direct line of sight is thus made horizontal. Bring the reflected image of the object into coincidence with the cross wire. The vernier reading will then be the required vertical angle.

For angles of elevation the face of the sextant would be turned to the left and for angles of depression the sextant would be turned over face to the right, the bubble of the hand level always being kept uppermost by turning the tube in the ring.

This method is not accurate because the hand level is not a sensitive instrument, but it permits the sextant to be used as a clinometer for measuring slopes and vertical angles.

USE OF THE SEXTANT.

The sextant is a valuable instrument for the reason that it is light and portable, and requires no fixed and rigid support. It, and kindred reflecting instruments, are the only ones that can be used for measuring angles, with accuracy, when the observer is moving. It is, therefore, always used at sea or on the water, where the motion of the vessel renders useless all instruments that require a fixed support. The compass is used at sea, but it measures horizontal angles only, and is not accurate.

The sextant is used:—

1. For astronomical observations on sea and land to determine latitude, longitude and time.

2. On the water for measuring horizontal angles to determine the position of the boat by the "three-point" problem. In hydrographic surveys this method is often used to locate soundings, buoys, rocks, shoals, etc.

3. As a surveying instrument for measuring the angles in a system of triangulation. It is especially suited for preliminary triangulation in determining suitable locations for triangulation stations, or for making a rough triangulation to control and check a hurried survey. The pocket sextant well serves this last purpose in connection with a hasty sketch survey of an extended "position."

4. The sextant may be used to run a traverse, but it is not well suited to this purpose, because it measures an angle in a plane that contains the two points observed, and not generally in a horizontal plane. Also, its measurements are limited to angles not exceeding 130 or 140 degrees. If, however, the

survey runs over level country, a sextant traverse may be made as accurately as a transit traverse, so far as angles or directions are concerned.

5. As a range finder or telemeter. Any instrument that measures angles accurately may be used in connection with a measured base to determine the distance from the observer to a distant point. The sextant may therefore be used for this purpose.

6. By substituting a hand level for the telescope, the sextant becomes a clinometer, and may be used to measure vertical angles and slopes.

7. Any problems in field geometry that depend on the measurement of angles may be solved by the use of the sextant.

THE CLINOMETER.

The clinometer is a hand instrument for measuring vertical angles and slopes with reference to the horizontal. It appears in various forms, all of which are based on one or the other of the two methods of determining a horizontal line, viz: the spirit level and the perpendicular to the plumb line.

The Abney clinometer is a typical form of the spirit level class. It consists of a sighting tube with a peep hole at the eye end and a cross wire at the other to determine the sighting line. To the tube is attached a vertical graduated arc, at the center of which is pivoted on a horizontal axis a spirit level having an index arm with a vernier to read the angle between the sighting line and the axis of the level tube. When the bubble is at the center of its tube the axis of the level is horizontal, and the angle indicated by the vernier is the angle made by the sighting line with the horizontal. A small mirror in the tube gives to the observer a view of the bubble tube while he is sighting at a distant point. By turning the level he brings the bubble to the middle of its tube and then reads the vernier for the angle of elevation or depression of the point observed. The sighting tube is rectangular in cross section, and therefore has a flat base. By placing the clinometer on any plane surface on the line of steepest slope, and bringing the bubble to the center of its tube, the inclination of the surface is given by the reading of the vernier.

Adjustment. The base of the sighting tube and the sighting line are made parallel by the manufacturer. When the base or the sighting line is horizontal and the bubble is at the middle of its tube, the vernier should read zero. Set the vernier at zero and place the clinometer lengthwise on a small piece of smooth board. Adjust the board by means of a small wedge, or by packing earth and sand under one end, so that the bubble shall stand at the middle of its tube. Mark the position of the clinometer by pencil lines on the board. Turn the clinometer end for end and place it carefully in the same position with respect to marks, taking care not to disturb the board. If the bubble remains at the center the adjustment is correct. If not, correct one-half of the displacement by the adjusting screws of the level tube and repeat the test. Other methods of adjustment may be readily devised.

The Watkins Clinometer is one that depends on a plumb bob for determining the horizontal reference line. It consists of a cylindrical metal case about 3 inches in diameter. In the axis of the cylinder there is pivoted a short pendulum which carries a light graduated circle centered at the pivot. The sighting line is fixed by a peep hole in one side and an open sight with a cross wire in the other side of the cylindrical case. A small concave mirror reflects a magnified image of the graduations of the circle to the eye when sighting at a distant point. That division of the circle whose reflected image coincides with the cross wire is the one to be read. The pendulum and its attached circle are so adjusted that when the sighting line is horizontal the reading of the circle is zero. The final adjustment is effected by a screw in the pendulum bob by means of which the position of the center of gravity may be changed.

In another form of this clinometer the graduated circle is fixed on the interior of the cylindrical case, and the pendulum provided at the center of the arc carries at its pivot a small vertical mirror which reflects an image of the graduations to the eye while sighting. When the sighting line is horizontal the zero mark is seen to coincide with the line of sight. When the sighting line is inclined an arc equal to twice the angle of inclination has passed in reflection through the mirror; therefore the divisions of the arc are figured with numbers equal to one-half the actual value of the arc.

A method of testing the adjustment of these pendulum clinometers may be readily devised.

The Slope Board. When a small drawing board is used in the field for plotting the survey or sketch as it progresses, the board itself may be made to serve the purpose of a clinometer in the following manner:

The back of the board is marked with a graduated semi-circle whose diameter is parallel with one of the edges of the board. The zero is at the middle of the arc and the degrees are numbered each way from the zero to 90 degrees. At the center of the arc a plumb line is suspended.

Hold the board in a vertical plane and sight along the diameter edge at the selected distant point. When the plumb line comes to rest hold it in place with the finger and read the arc at the point where the line cuts it. The angle thus read is the inclination of the line of sight, plus (+) if above the horizontal and minus (—) if below.

Sketching cases are generally provided with the slope board attachment.

PLATE B.

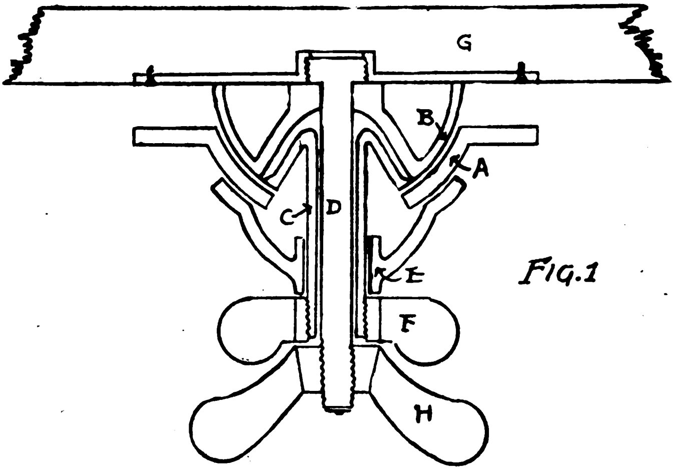


FIG. 1

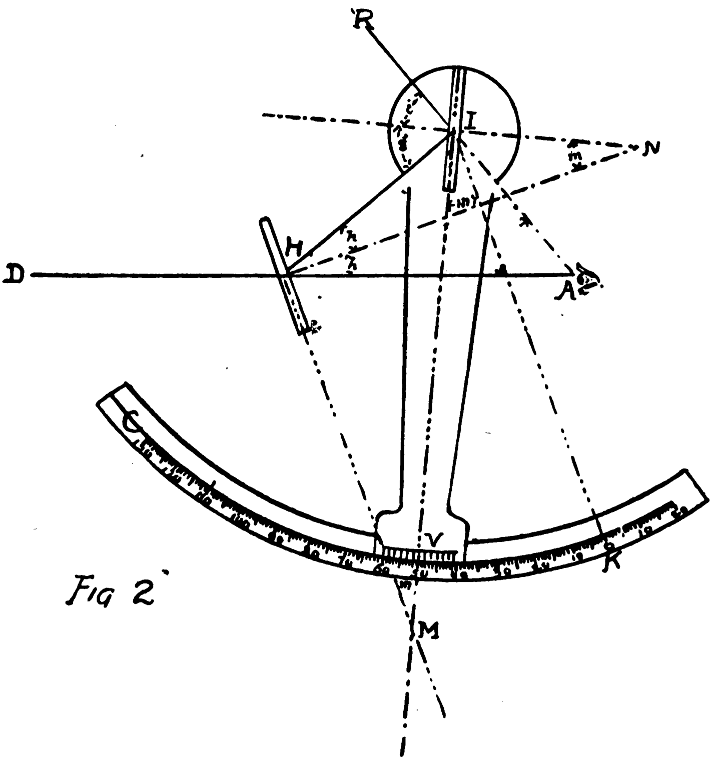


FIG 2

PLATE C.

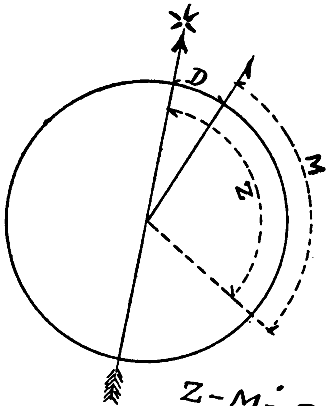


Fig. 1.

Z - M = D

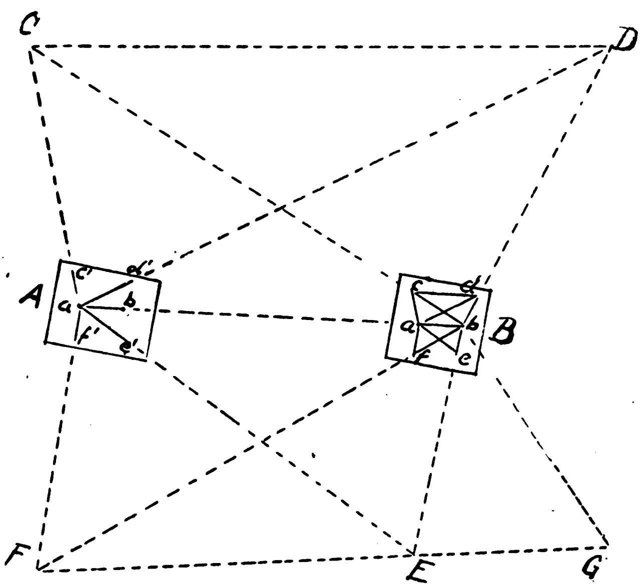


Fig. 2

CHAPTER IX.

THE PLANE TABLE.

The plane table is a surveying instrument consisting of a drawing board supported on a tripod, and a ruler to which a pair of sights or a telescope is attached. It is used in the field for projecting the lines and points of a survey directly upon the drawing, and thus it eliminates the measuring of horizontal angles and the use of the protractor. The transit or theodolite, the sextant and the compass are used to measure angles, and thus determine directions. The plane table is used to determine directions without measuring angles. Since its purpose is the same, it is generally classed with the angle measuring instruments.

DESCRIPTION.

The board should not be larger than 24x30 inches. A large board is difficult to keep level, and is heavy to carry. A smaller board, say 18x22 inches, is better if it fulfills the purpose of the survey. The board should be made of light material (white pine is best) and should be so constructed as to prevent warping and splitting.

The drawing paper should be fastened to the board by means of screws with flat heads. Metal sockets for these screws are let into the wood at the corners of the board, and at these points the board is countersunk so that the top of the screw head shall be flush with the surface of the board. The ruler can then be placed anywhere on the board without interference. Other means of fastening the paper are thumb tacks and spring clips, but these interfere with the ruler in some positions. Thumb tacks also mar the board. Rollers at the ends of the board are sometimes used for fastening and stretching the paper.

The board rests on a leveling head, to which it is fastened by screws with a connection that permits the board to be revolved in its own plane or to be clamped in a fixed position. Various forms of leveling heads are used for the plane table. The most accurate method of leveling is by means of leveling screws like those of the transit.

The Johnson plane table movement is compact and simple. (Plate B, Fig. 2.) The top of the tripod is a spherical cup (A) with a large hole in the bottom. The board (G) is fastened to a similar cup (B) which nests into the first and forms a universal joint which permits the board to be leveled and turned in azimuth.

These motions are clamped by means of a double spindle (C D) that passes through the hole in the lower cup. A washer (E) and wing-nut (F) on the outer spindle (C) clamp the leveling motion, while leaving the board free to turn in azimuth. A wing-nut (H) on the end of the inner spindle (D) clamps the azimuth motion. The large spherical surface of contact at the joint gives great rigidity and firmness to the connection. A slow motion tangent screw cannot be applied to this device.

The alidade of the plane table consists of the ruler with its telescope or sights. The telescopic alidade should be used for accurate work. The ruler should be broad enough (at least three inches) to provide a firm support for the telescope. A brass column firmly fixed to the ruler carries the telescope on a horizontal axis that is perpendicular to the edge of the ruler. The line of collimation of the telescope, therefore, revolves in a vertical plane that is parallel to the edge of the ruler. The telescope is like that of the transit and has stadia wires, attached level, and vertical circle.

In some instruments the plate levels and compass needle or declinator are attached to the ruler, but this increases the weight of the alidade. The alidade must be lifted and moved whenever a new sight is to be taken, while the compass and plate levels are needed only occasionally. The latter should therefore be placed on a separate plate and removed from the table when not in use. For reading the bearings of lines the compass must have a full circle, and such a compass cannot conveniently be mounted on the ruler. Most instruments, therefore, have a compass in a square case, with the plate levels set at right angles to each other along its edges. The zero line of the compass is parallel to two of the edges of the case. To read the bearing of any line on the board, place one of these edges on the line and read the compass. To read the bearing of any direction line sight the telescope in the desired direction and then place the edge of the compass against the edge of the ruler. The compass reading will give the bearing of the line.

The plane table has parts analogous to every part of the transit and used for the same purposes. The telescope, attached level, stadia, vertical circle, compass, plate levels, leveling head, clamps, tangent screws and tripod are practically the same for both instruments. The board of the plane table corresponds to the horizontal circle of the transit, and the ruler of the plane table corresponds to the vernier plate of the transit. The adjustments of the plane table are therefore similar to those of the transit.

ADJUSTMENTS.

1. To adjust the plate levels when attached to the declinator, lay the declinator on the table and bring the bubbles to the centers. Draw pencil lines at the edges of the declinator. Reverse it 180° and replace it at the same marks. If the bubbles return to the centers the levels are in adjustment. If the bubbles are displaced, bring them back over one-half of the displacement by re-leveling the board. This will make the board level. Then bring the bubbles to their centers by the adjusting screws. Repeat the test, and make further adjustment if necessary.

Otherwise, level the table carefully by means of the telescope level by placing the ruler direct and reversed in two positions at right angles to each other. When the board is level place the declinator on it and bring the bubbles to their centers by means of the adjusting screws.

The telescope level being more sensitive than the plate levels, the board may be very accurately leveled by its use, but the plate levels are more convenient for leveling the board when they have been properly adjusted.

2. To adjust the line of collimation so that it will generate a plane in its revolution on the horizon-

tal axis. This adjustment is identical with the like adjustment of the transit. It must be remembered, however, that in reversing the telescope after it has been plunged the reversal must be made about a point on the board that lies directly beneath the point of intersection of the axis of the telescope and the horizontal axis.

3. To adjust the horizontal axis. This adjustment is identical with the third adjustment of the transit. The same precaution is necessary in this as in the previous adjustment.

4. To adjust the telescope level so that its axis shall be parallel with the line of collimation. This adjustment is made in the same manner as the like adjustment of the transit. Otherwise, as follows: Set up the plane table and level it carefully, using the telescope level. Sight at a graduated rod held vertically on a peg at a measured distance, say 100 feet, and read the rod. Mark the position of the ruler by pencil lines, and then reverse the alidade and set it at the same lines in reversed position. Read the rod held on a peg at the same distance (100 ft.) in the opposite direction. The difference of these readings will be the difference of elevation of the two pegs. Move the plane table and set it up, carefully leveled, close to one of the pegs. Set the alidade so that a sight may be taken backward through the telescope at a rod on the near peg, and forward, without moving the alidade, at a rod on the far peg. The backward sight must be taken with the eyepiece very close to the rod, the eye being at the object glass. Bring the telescope bubble to zero, and by the backsight determine the H. I. with reference to the near peg. To this H. I. add algebraically the differ-

ence of elevation of the two pegs as already determined and set the target on the rod at the corresponding reading. Sight forward at this rod on the far peg. By bringing the cross wires to bisect the target accurately, the line of collimation is made horizontal. Then with the adjusting screws of the level tube, bring the bubble to zero.

5. To adjust the vernier of the vertical circle. Having made the fourth adjustment as just described, adjust the vernier without moving anything else, so that it shall read zero. If the vernier is not adjustable a slight modification of the fourth adjustment is necessary as follows: Before taking the back and foresights at the near and far rods respectively, set the vernier at zero and adjust the telescope level so that its bubble shall read zero. Then in sighting at the target on the more distant rod, effect the bisection by adjusting the cross wires without moving anything else.

It is sometimes inconvenient to sight backward through the telescope at the near rod. If so, simply measure the height of the center of the eyepiece above the peg to get the H. I., using the rod or tape.

The methods just described for making the fourth and fifth adjustments of the plane table are equally applicable in the like adjustments of the transit.

6. The vertical plane in which the telescope revolves should be parallel with the edge of the ruler. There is no adjusting motion by means of which this parallelism may be effected, but it may be tested as follows:

Place the alidade on the plane table and sight along the edge of the ruler. Set a stake and tack or other precise mark in the line of sight at a convenient distance, say 100 or 200 feet. Sight through the telescope toward the same mark and mark a point in the new line of sight. The horizontal distance between the two marks should be the same as the horizontal distance between the edge of the ruler and the line of collimation of the telescope. If the latter lines are in the same vertical plane the two marks should coincide.

So far as the plane table plot is concerned it would make no difference whether the edge of the ruler were parallel to the vertical plane through the line of collimation or not. In fact, it might make any angle whatever with that plane without affecting the accuracy of the plot, but if it is desired to read the compass bearing of any line of the survey, the edge of the ruler and the vertical plane containing the line of collimation should be parallel. It is always necessary to draw a meridian on the map, that is, to determine a line whose bearing is zero. If the edge of the ruler and the line of collimation of the telescope do not lie in parallel or coincident vertical planes, a magnetic meridian may still be drawn on the plot as follows:

Place the compass on the plane table and turn it till the needle reads zero. Sight along a meridian edge of the compass case and note or set a distant point in the line of sight. Mark a point on the paper at the sighting edge of the compass. Pivot the ruler at this point and site the telescope at the same distant point. A line drawn along the edge of the ruler will be a magnetic meridian of the plot. By the same method the bearing of any line of the survey may be determined, but the inconvenience of this method as compared with the simple method that may be used with a properly adjusted alidade is apparent.

A sight alidade is a ruler having sight leaves like those of the surveyor's compass, one at each end. The only adjustment is that for verticality of the sight slits made by sighting at a plumb line.

PLANE TABLE SURVEY.

When a telescopic alidade is used the principles and methods employed are similar to those that obtain in a transit and stadia survey. When a sight alidade and chain are used the principles and methods are similar to those that apply in a compass and chain survey, except that in both cases, directions and distances are plotted at once on the paper and are not recorded.

With the telescopic alidade the party should consist of one surveyor, two stadia rodmen, and one axeman. With the sight alidade there will be required one surveyor, two chainmen, and one rodman. In the latter case the surveyor reads vertical angles with a clinometer. Better progress will be made if a draughtsman is added to the party as an assistant to the surveyor. The latter points the telescope or sights, reads the stadia rod or directs the chaining, and reads the vertical angle, while the draughtsman lays off the distance to scale on the drawing, plots the determined point, computes its elevation, interpolates contour points and sketches the located objects and features.

A plane table survey should be controlled by triangulation, which may have been made with transit or sextant, or which may be made with the plane table, as follows:

Select a suitable base line on ground that is unobstructed and fairly level. Place station marks at the ends of the base and determine the horizontal distance between them by measuring carefully with a steel tape. Select triangulation stations which form fair shaped triangles and which are mutually visible along the sides of the triangles.

Call the base stations A and B respectively and other stations, C, D, E, etc.

Draw a line on the plane table paper to represent the base line, selecting such a scale and such a position on the paper as will cause the entire survey or any desired portion of it to fall within the limits of the paper. The map stations will here be called a, b, c, etc., to distinguish them from the corresponding ground stations, A, B, C, etc.. (Plate C.)

Set up the plane table at station A, with the line a b on the paper lying in the direction A B, and the point a on the paper directly over the station A on the ground. Level the board carefully and complete its orientation by placing the edge of the ruler on the line a b, and sighting through the telescope or sights at B, and turning the board so that the mark at B is bisected. Clamp the board in this position.

Pivot the edge of the ruler at a and sight in succession at C, D, E and F, drawing at the ruler's edge for the several pointings the lines a c', a d', a e' and a f'.

Move the plane table to station B and set it up with point b over station B and line b a pointing toward station A. Orient accurately by using the alidade and sighting the line b a at the station mark at A. Pivot the ruler at the point b, and sight in succession at stations C, D, E, and F, drawing the line for each pointing. The intersections of these lines with the corresponding lines drawn from a are at c, d, e and f, and are the plotted positions of stations C, D, E and F, respectively.

To check the accuracy or the plot, set up the plane table at station C, with point c of the plot over the station mark. Orient by a backsight of the line c a at A, and verify the orientation by a backsight on the line c b at B. Pivot the ruler at C, point at D and draw the line. It should pass through the point d on the plot.

Move the plane table to station D, orient by backsights at B and A, pivot the ruler at d, sight at C, and draw the line. It should pass through the point c on the plot.

If the three lines, b d, a d and c d, do not meet in a common point they will form a small triangle whose center of gravity may be taken as the corrected position of the plotted point d. A similar correction may be made at c, and a like check and adjustment may be made at e and f by occupying the corresponding stations.

Any two of the determined stations may be used as the ends of a new base from which other triangulation stations may be plotted, and thus the system of triangles may be made to extend over any desired area.

The observations at each station include the reading of vertical angles to adjacent stations. The horizontal distance between any two stations is scaled from the plot, and this distance, multiplied by the tangent of the corresponding vertical angle, gives the difference of elevation between the stations. A method of adjusting these differences of elevation will be given in a subsequent chapter. When adjusted, an elevation may be assumed for any station, and the elevations of all others may be computed.

Every station should be marked with a staff and a distinguishing flag when not occupied.

To draw a magnetic meridian, orient the board carefully at any station by sighting at adjacent stations, place the declinator on the board and bring the north end of the needle to a zero reading, draw a line along the edge of the declinator parallel to the needle.

To draw a true meridian, orient the board carefully at any station and sight the telescope at Polaris at time of elongation. Draw the line so determined and lay off on it from the station point any convenient length as 10 inches. Multiply by the tangent of the star's azimuth and lay off the result in inches on a perpendicular at the end of the line toward the meridian. Draw the true meridian through the point so determined and the station point.

The survey is completed by running a network of traverses, each of which begins and ends at a triangulation station.

TRAVERSING WITH THE PLANE TABLE.

Set up the plane table at a triangulation station and orient by sighting at an adjacent station with the alidade on the line that joins the corresponding map stations. Pivot the ruler at the station point and sight at a rod held by the rodman at selected critical points. The vertical angle is read on a mark whose height on the rod is equal to the height of instrument above the ground. The distance is measured by the stadia or by chaining. The plotting scale may be marked on the edge of the ruler and its zero held at the pivot station. At each sight lay off the measured distance and plot the point. Compute its elevation and mark it at the point. Draw the object or feature thus located, and when all desired points have been plotted, interpolate contour points and draw the contours, shaping them by observation of the shape of the ground. Finally check the orientation by sighting again at a known station, select a good position for a new traverse station and plot it by careful determination of its direction, distance and elevation.

Set up the plane table at this new station (No. 1), orient by a backsight at the preceding station, checking the vertical angle and also the distance, if measured by stadia. Proceed to fill in details by side shots as at the preceding station, and finally verify the orientation and observe and plot station 2. The foregoing operations are repeated at successive stations, and the traverse is so run as to close on a triangulation station, which becomes the last station of the traverse.

The position of the last station as determined by the traverse will not in general coincide with its true position as determined by triangulation, and the discrepancy is the error of closure of the traverse. The correction is usually all made in the last course, which should, therefore, be as long as possible. To distribute the error throughout the traverse would require the erasure of all the previous work. If the error is large, indicating that a "mistake" has been made, the traverse must be erased and done over again. At the closing station set up the plane table and orient by a backsight at the preceding traverse station; then verify the orientation by sighting at the initial station or some other triangulation station. The error in orientation that has accumulated while traversing is thus determined.

Other traverses are run between the triangulation stations until all of the ground has been covered and all its details filled in on the map. The errors that accumulate in any traverse are thrown out at the closing station and do not affect subsequent work.

It is often convenient to begin or end a traverse at an unknown point or to check the position of an intermediate station of a long traverse. Such points may be determined and plotted by *resection* from three or two known stations that are visible from the station occupied.

Resection from three known stations is an application of the "three point problem." Fasten a piece of tracing paper on the board over the map and assume any point to represent the station occupied. Pivoting the ruler at this point, sight in succession at the three known stations and draw the corresponding lines. Shift the paper on the map so that the three lines shall pass through the plotted positions of the three corresponding stations. Plot the station occupied by pricking through at the assumed point. This method fails when the station occupied lies on or near the circle that passes through the three known stations.

If the unknown station can be so selected as to be in range with two known stations while a third is visible, as at G, Plate C, in range with E and F, the plane table may be oriented at G by sighting the line e f at E and F. Then pivot the ruler at the point b and sight at B. Draw a line back from B to its intersection with f e prolonged. This intersection will be g, the plotted position of the station occupied, G.

If at any unknown point the plane table can be properly oriented, the point may be plotted by resection on two known stations. Thus at G, if stations E and B alone are visible, the board may be oriented by placing the declinator on the magnetic meridian line and turning the board till the needle reads zero. Then pivot the ruler at e, point at E and draw the line e g. Pivot at b, sight B, and draw b g. The intersection g of these two lines will be the plotted position of station G. Orientation by the compass is not accurate, and this method cannot be relied upon for good results.

Resection on two known points may be made without the compass by using an auxiliary base line at the unknown point. Disregard former references to Plate C and suppose now that C and D are two triangulation stations visible from an unknown point B. Select another station as A, also unknown, and use A B as a base line. Its length is not measured. Set up the plane table at A and place on the board a piece of tracing paper. Assume a point, a, over the station mark A, to represent the station occupied. Pivot the ruler at a and sight at C, D and B, drawing a c′, a d′ and a b, marking any point as b to represent station B, distance unknown and not to scale. Set up at station B with the point b over the station mark. Orient by sighting the line b a at A, pivot at b and sight at C and D, drawing b c and b d. They intersect the corresponding lines drawn from a in c and d. Draw c d.

There has thus been plotted on tracing paper the quadrilateral a b d c representing the similar quadrilateral A B D C on the ground. Shift the tracing paper so that point d on the paper shall coincide with the plotted station d on the map and the line d c on the paper with the line d c on the map. Orient the board by sighting the line b d on the paper at station D, remove the tracing paper and plot b on the map by resection on C and D.

Station A may also be plotted either by using the tracing paper to lay off the angles d c a and c d a on the map, or by setting up again at A orienting on B, and resecting on C and D.

If A and B are traverse stations near the middle of a long traverse, a check has thus been applied by which accumulated errors may be thrown out and not transmitted throughout the remainder of the traverse.

The operations of inking and finishing the map are the same as the like operations of a transit survey.

CHAPTER X.

MEASUREMENTS OF DISTANCE.

The distance between two points is the length of the right line joining the two points, as A B (Plate 6, Fig. 3).

The horizontal distance between two points is the length of a horizontal line limited by verticals through the two points, A′ B or A B′.

The vertical distance between, or difference of elevation of two points, is the length of a vertical line limited by horizontals through the two points, A A′ or B B′.

In surveying, horizontal and vertical distances are required, and may be determined in two ways.

First, by direct measurement of the required distances. Thus for the horizontal distance from A to B, measure the length of the line A′ B, or what is the same thing, measure in succession the lengths between intermediate verticals of the lines c d, d′ e, e′ f and f′ g. Their sum will be the required horizontal distance from A to B. For the vertical distance from A to B measure the length of the vertical A A′, or, what is the same thing, measure in succession the lengths between intermediate horizontals of the lines + A c, + d d′, + e e′, + f f′, and — g B. Their algebraic sum will be the required vertical distance from A to B.

Second, by measuring one side and one angle of the vertical right triangle whose base is the horizontal distance between the two points and whose hypothenuse is the line joining two points, and solving the triangle for the required parts. Thus (Plate 7, Fig. 5), (1) measure the inclined line $AB = b'$ and the angle of inclination C. Then for the horizontal distance, $b = b' \cos c$, and for the vertical distance, $a = b' \sin c$.

Or (2) measure the horizontal distance $A'B = b$, and the angle C. Then, for the vertical distance, $a = b \tan C$.

Or (3) measure the vertical distance $AA' = BB' = a$ and the angle of inclination, C. Then, for the horizontal distance,

$$b = a \text{ cotan } C.$$

This last mode of measurement (3) is seldom used in surveying, but the principle finds application with corrections for refraction and curvature, in the depression angle range finder used in connection with sea coast fortifications.

The instrument is mounted at B (Plate 7, Fig. 1), the height BB' above sea level being accurately known. The distance, d from B' to some point as A on the surface of the sea is required.

The effect of refraction is to raise the apparent position of a point observed by an amount that is proportional to the square of its distance from the observer, and that is equal to about one-seventh of the departure of the earth's curved surface from the horizontal. Therefore the apparent surface of the sea is a spherical surface whose diameter is $\frac{7}{6}$ times the earth's diameter, and the point A on the true surface will appear to be raised to A′ on the apparent surface. The apparent surface only need be considered. The formula $b = a \cot C$ gives the distance B′ C to which must be added C A″ to give the required distance *d*. But $C A'' = C'c \text{ cotan } C$, and if D represent the diameter of the apparent surface, then, since a tangent to a circle through a given point is a mean proportional between a secant line and its external segment, there results (Plate 7, Fig. 2):

$$C'C : b :: b : C'C + D$$

$$\text{whence } C'C = \frac{b^2}{C'C + D} = \frac{b^2}{D} = \frac{a^2 \cot^2 C}{D}$$

because C′ C is very small as compared with D, and may be neglected in the denominator of the second member.

$$\text{Then, } C A'' = \frac{a^2 \cot^3 C}{D}$$

$$\text{and } d = b + C A'' = a \text{ cotan } C + \frac{a^2 \text{ cotan}^3 C}{D}$$

$$= a \text{ cotan } C \left(1 + \frac{a}{D} \text{ cotan}^2 C\right).$$

The instrument is so constructed as to solve this formula mechanically and give a direct reading of the distance, *d*. It must be adjusted for stage of tide which affects *a*, and for varying refraction which affects D. When properly adjusted, distances may be read with a fair degree of accuracy up to 5,000 or 7,000 yards.

Direct measurements of distances are made with rods or bars of metal or wood, with chains of metal, with tapes of metal or of woven fiber, and with the stadia. Less accurate measurements may be made by pacing, or by counting the revolutions of a wheel, or by moving over the line at a known rate and taking the time, or by using a pole or a piece of rope, wire, or cord.

In every case the length of the rod, bar, chain, tape, pace, pole, rope, wire or cord, the circumference of the wheel, or the distance covered in a unit of time, as the case may be, must be accurately known in terms of some standard unit. The units of measure commonly used are the inch, foot, yard, 100 feet, link, chain or meter.

Bars and rods of metal or wood in lengths of ten to fifteen feet, or three or four meters, have been used for very careful and elaborate measurements of base lines for important surveys. It has been found, however, that the steel tape in lengths of 100 to 400 feet, with the necessary corrections for temperature, pull, sag and slope, is just as accurate as the wooden or metal bar. It is also much more rapid and economical than the bar for measuring long lines with accuracy, and it has therefore almost entirely displaced the latter for this purpose. A graduated bar or rod is often used for measuring short distances, as in laying out a building or other structure, but in this case the steel tape is more convenient and more accurate.

In measuring vertical distances or differences of elevation the graduated rod holds its place as the most suitable instrument. The tape or chain cannot be held vertical without a point of support overhead, but a rod can be held vertically with a point of support on the ground. Rods used for vertical measurements in surveying are called *level rods*, but this is a misnomer. They are never used in a level position, but always in a vertical position, and "vertical rod" would be a better name than "level rod." the term "level rod" is, however, too well established to be displaced.

In order to measure with a graduated rod the differences of elevation of points on the earth's surface, it is necessary first to establish a horizontal line of known elevation, and then measure vertically from this line to the required points. A horizontal line is established by means of a level.

THE WYE LEVEL.

The Wye level is an instrument designed to establish with great accuracy a horizontal line of sight. It has the following parts: The telescope A (Plate 7, Fig. 3), the level tube B, the wyes *c c* supporting the telescope, the bar D which has a vertical spindle fitting in the socket of the leveling head E, the clamp and tangent F, and the tripod G.

The telescope is similar to, but generally longer than that of the transit. Its line of collimation is defined by adjustable cross wires. Stadia wires are not provided unless specially ordered.

The level tube is attached to the under side of the telescope tube with adjustable connections providing for a slight vertical motion at one end and a slight horizontal motion at the other end. The level tube is long and is carefully ground and graduated. A very slight deviation of its axis from the horizontal is at once made apparent by a large displacement of the bubble from its zero position.

The telescope is supported in the wyes at two rings accurately turned to the same diameter, so that a reversal of the telescope, end for end, in the wyes will not affect the position of the axis of the rings.

The wyes are supported on the bar. One support (often both) is adjustable, and provides for a slight vertical motion by means of which the axis of the wyes may be made perpendicular to the vertical axis of the instrument, that is, to the axis of the spindle.

The spindle is fixed perpendicularly to the bar. It is turned to a conical form and fits in the socket of the leveling head.

The leveling head is similar to that of the transit, but it must always have four leveling screws and not three. The purpose of the leveling head is to make the axis of the spindle vertical, so that the line of collimation shall remain horizontal when turned on the spindle.

The tripod provides a rigid support for the instrument. A plumb line is not necessary in setting up the level, because it is not essential that the level be set exactly over any particular point on the ground.

ADJUSTMENT OF THE WYE LEVEL.

It is required of the level that the line of collimation of the telescope may readily be made horizontal and that it shall remain horizontal, or generate a horizontal plane, when revolved on the spindle.

To fulfill these requirements, the axis of the spindle must be vertical and the line of collimation of the telescope must be perpendicular to that axis. The axis of the spindle can always be made vertical by means of the leveling screws and the bubble tube.

The adjustment of the wye level therefore consists in making the line of collimation of the telescope perpendicular to the axis of the spindle. The bubble tube is also adjusted so that its axis shall be parallel to the line of collimation.

The principal adjustment of the wye level may be effected by the following operations:

1st. Make the axis of the spindle vertical.

2d. Make the axis of the wyes horizontal.

3d. Make the line of collimation of the telescope parallel to the axis of the wyes.

4th. Make the axis of the bubble tube parallel to the axis of the wyes.

First Step. — To make the axis of the spindle vertical. Set up the level and turn the telescope to a position parallel to the line joining two opposite leveling screws. With these screws bring the bubble to a center. Revolve 180° on the spindle and note the new position of the bubble. With the same leveling screws bring it back over a space equal to one-half its total displacement. This places the axis of the spindle in a vertical plane perpendicular to the axis of the bubble. Turn the telescope 90° to a position parallel to the line joining the other pair of leveling screws and repeat the operations just described. This will make the axis of the spindle vertical. Test its verticality by turning the telescope back to its first position, direct and reversed, and again to the second position, direct and reversed, making further corrections if necessary. When the bubble remains fixed for all positions of the telescope, the axis is vertical. This fixed position of the bubble is not necessarily at the center.

Second Step.—To make the axis of the wyes horizontal. The axis of the wyes is the axis of a cylinder that is tangent to the wyes at the points touched by the rings of the telescope.

Having made the spindle vertical, note the exact position of the bubble. Open the clips and carefully reverse the telescope end for end in the wyes. Observe the displacement of the bubble and correct one-half of this displacement by adjusting the wye supports. This will make the axis of the wyes horizontal. Repeat the test, and make further corrections if necessary.

Third Step.—To make the line of collimation of the telescope parallel to the axis of the wyes.

Sight through the telescope and direct the cross wires upon a well defined point distant 350 to 500 feet. Clamp the level and secure exact bisection of the point by means of the tangent screw and leveling screws. With the clips loosened, revolve the telescope 180° on the axis of the wyes so that the level tube shall be on top. If the cross wires have moved from the point at which they were first directed and now bisect a new point, adjust them by means of their adjusting screws so that their intersection shall fall midway between the two points observed. This will make the line of collimation parallel to the axis of the wyes. Repeat the test by revolving the telescope to its normal position. If the intersection of the cross wires does not move on the image during the revolution, this part of the adjustment is correct.

Fourth Step.—To make the axis of the bubble tube parallel to the axis of the wyes.

First, bring the bubble to the center by means of the vertical adjusting screws at one end of the bubble tube. This will make the axis of the bubble tube horizontal, and therefore parallel to the horizontal plane that contains the axis of the wyes.

Second, turn the telescope slightly in the wyes (only 10 to 15 degrees from its normal position). If the bubble moves bring it back to the center by means of the horizontal screws at one end of the bubble tube. This will make the axis of the bubble tube parallel to the vertical plane that contains the axis of the wyes. Having been made parallel to two planes that contain the axis of the wyes, the axis of the bubble tube will be parallel to that axis

In practice, the fourth step of the adjustment is very important, because it is convenient that the central position of the bubble shall indicate horizontality of the line of collimation. Any other fixed and known position of the bubble in its tube would serve as well, but the central position is clearly marked on the tube, and it is therefore easy to tell when the bubble has this position.

REMARKS ON THE ADJUSTMENT.

Any fixed position of the bubble in the tube might be used for determining the horizontality of the line of collimation. It is only for convenience that the central position is selected and the bubble adjusted to that position when the line of collimation is horizontal.

If the line of collimation and the axis of the bubble tube are parallel, the line of collimation can be made horizontal by bringing the bubble to the center no matter what may be the position of the axis of the spindle, but in order that the line of collimation may remain horizontal when turned on the spindle to any other desired position, the axis of the spindle must be vertical. It is impossible, with a sensitive level, to make the adjustment so perfect that there will be no motion of the bubble. Whenever the level is turned to a new direction the bubble should be examined and brought to the exact center by means of the leveling screws.

The adjustments of telescope for centering the object slide and eyepiece and for verticality of the vertical wire are the same as the like adjustments of the transit.

The *Architect's Level* is smaller than the engineer's wye level and has a horizontal graduated circle for laying off angles on the ground. The *Dumpy Level* has no wyes, and the telescope is rigidly fixed in its supports. It is not capable of such careful adjustment as the wye level, but is less liable to get out of adjustment.

The *Hand Level* is a simple sighting tube with a small spirit level set over an opening in the top of the tube. A small mirror in the tube reflects an image of the bubble to the eye of the observer who is thus enabled to hold the tube in a horizontal position while sighting. It is used only in hasty sketching, and not in accurate surveying.

The level serves to establish a horizontal line of sight to be used as a reference line from which vertical offsets may be measured, by means of a graduated rod, to points on or near the earth's surface.

LEVEL RODS.

Level Rods are usually graduated to show feet, tenths and hundredths or to show meters and centimeters. By estimation or else by using a sliding target with a vernier, heights may be read to the thousandths of a foot or to the nearest millimeter. The zero of the rod is at its lower end.

Rods are called speaking or self-reading rods when their graduations are so plainly marked and numbered that they may be read by the observer when he sights through the telescope. The point on the rod that is cut by the horizontal wire is the point to be read.

The target rod is provided with a sliding target on which is painted a device in contrasting colors that clearly defines a horizontal line and a vertical line through the center of the target. The observer signals to the rodman to move the target up or down on the rod till the horizontal line on the target coincides with the horizontal wire of the telescope. The rodman then reads the rod at the point cut by the line. An opening in the target uncovers the marks on the rod, and a vernier at the edge of this opening reads to the nearest thousandth of a foot or to the nearest millimeter.

A folding rod is one that is made in two pieces that are tongued and grooved together lengthwise and thus may be extended by sliding one part beyond the other.

The rods furnished by instrument makers have in each case one or more of the features that have been described.

The simplest is the plain self-reading rod, and for this the manufacturers need not be depended on since it is easy to make one. A board of well seasoned white pine 1" x 4" x 12' is used. Plane it smooth and straight and rub down with sandpaper. Apply three coats of pure white lead paint tempered thin, and rub down with sand paper when the first coat is dry and again when the second coat is dry. When the third coat is dry, stud one end with large tacks closely placed or shoe it with strap iron to prevent wear. Beginning at this end lay off feet and tenths, and mark the divisions by cross lines drawn with a square. Paint with black a suitable device that will plainly define the divisions into feet and tenths of a foot. The device given for the stadia rod (Plate 1, Fig. 3) is suitable. Otherwise, alternate tenths may be painted in bands of black, with a distinguishing mark at every foot, and a different mark at every half-foot division. The feet should be plainly numbered from the bottom upward. The tenths also are sometimes numbered.

The Philadelphia Rod is a folding rod that may be used either as a self-reading rod or as a target rod. The divisions into feet, tenths and hundredths are so plainly marked that they can be read through the telescope by the observer. The closed rod reads from the bottom upward to seven feet. When fully extended the graduations are continued on the extended rod up to thirteen feet. When the target is used it is read up to seven feet on the face of the closed rod. For a longer rod, the target is clamped at the seven foot mark and raised to the required height by extending the rod. The extended rod is read by means of a scale on the back, which is numbered downward from seven to thirteen feet. The index of this scale is fixed near the top of the front part of the rod.

The New York rod is a folding target rod. The divisions into feet, tenths and hundredths are made by fine lines that cannot be seen at a distance even with the telescope. The target is therefore necessary. It is provided with a vernier reading to thousandths of a foot. The closed rod reads from the bottom upward to 6.5 feet. For a longer rod the target is clamped at the 6.5 ft. point and raised to the desired height by extending the rod. The extended rod is read by means of a scale numbered downward from 6.5 feet to 12 feet on the side of the extended portion. The index and vernier for this scale are marked near the top of the front part of the rod.

Other forms of folding and target rods are manufactured, but those that have been described are the ones generally used for measuring vertical distances.

PLATE VII

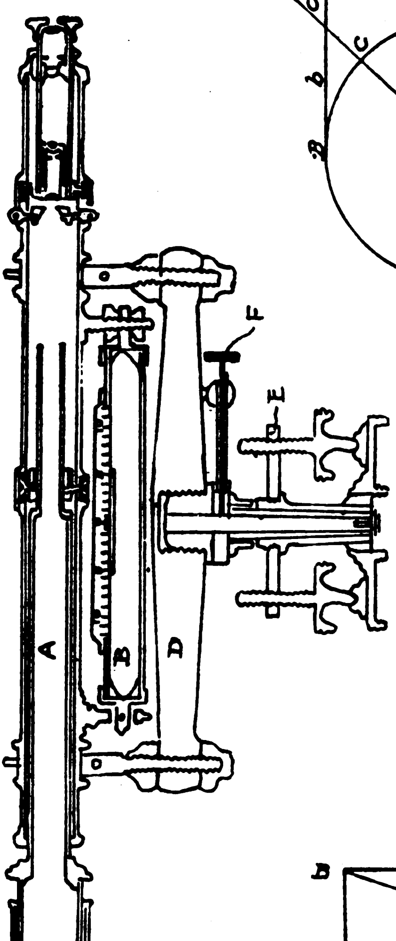


Fig 3

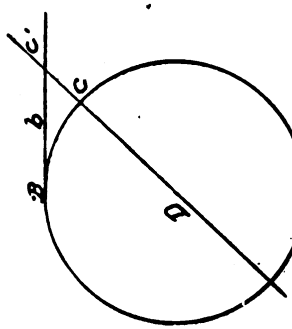


Fig 2

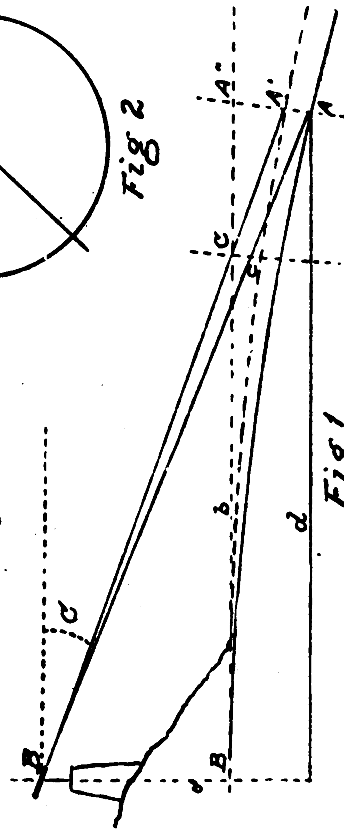


Fig 1

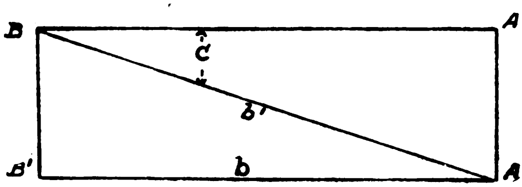


Fig. 5

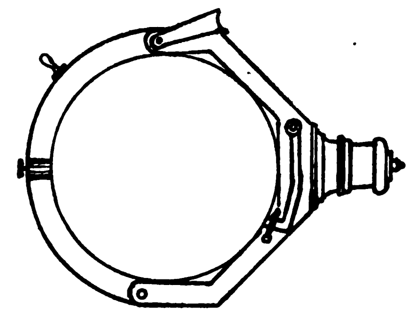


Fig. 4

CHAPTER XI.

USE OF THE LEVEL ROD AND LEVEL.

The principal adjustment of the level should be tested at the beginning of each day's work. With practice this becomes a matter of a few minutes only.

To set up the level, plant the tripod firmly and turn the telescope to a position parallel to the line joining two opposite leveling screws. With these screws bring the bubble to the center. Turn the telescope 90° and with the other pair of leveling screws bring the bubble again to the center. Repeat these operations in both positions and reverse the telescope in each position. If the bubble does not remain in its central position when reversed, an error of adjustment is shown. If the error is very small no correction should be made, but the bubble should be brought to the exact center by means of the leveling screws for every sight. If the bubble moves several divisions from the center when reversed, bring it back half way by the leveling screws and the other half by adjustment of the wye supports. In this way the second step of the adjustment of the level is tested at every set up.

The surveyor should always have in his pocket adjusting pins for any instrument he is using.

The sunshade should be used habitually to protect the objective from the direct rays of the sun, not only to avoid the glare of unnecessary light, but also to prevent unequal heating of the lens and consequent distortion.

The object glass must always be carefully focused for each sight, so that there shall be no apparent motion to the cross wires when the eye is moved at the eyepiece.

The bubble must be watched and brought exactly to its zero position whenever a sight is taken.

When the level is set up and leveled, a line of sight that coincides with the line of collimation is a horizontal line, and may be used as a reference line from which vertical offsets may be measured with the rod to points on or near the ground.

When the telescope is turned on its vertical axis the line of sight generates a horizontal plane which may be used as a plane of reference. From any point in this plane vertical offsets may be measured with the rod to points on or near the ground.

The difference of the rod readings or vertical measurements at any two points is the difference of elevation of the two points.

It is usual and convenient to consider the first point as an origin and to take the first rod as an upward or positive measurement from this origin to the line of sight, and the second rod as a downward or negative measurement from the line of sight to the second point. Thus (Plate 8, Fig. 1),

Let A be the first point or origin.

B the second point.

C the position of the level, equi-distant from A and B, but not necessarily in the line joining them.

C D and C E, the horizontal lines of sight to the rods held at A and B respectively.

a = rod reading at origin A.

b = rod reading at another point B.

d = difference of elevation or vertical distance from A to B.

Then a — b = d.

If a is greater than b, d is positive, which means that there is a rise from A to B or that B is the higher point. If a is less than b, d is negative, which means that there is a fall from A to B, or that B is the lower point, and in general, a *positive* difference of elevation *from* one point *to* another means that the second point is above the first and a *negative* difference of elevation *from* one point *to* another means that the second point is below the first.

EXAMPLES.

1.	Rod on point A....................	+ 9.563
	Rod on point B....................	— 4.382
	Diff. of elev. A to B	+ 5.181
	Hence B is 5.181 feet above A.	
2.	Rod on point C....................	+ 3.822
	Rod on point D	— 8.547
	Vertical distance C to D	— 4.725
	Hence D is 4.725 feet below C.	

If the difference of elevation be stated in the opposite direction, as from B to A, from D to C, the signs must be reversed.

A rod reading on a point that is assumed as an origin or on a point whose elevation is known, taken for the purpose of determining the height of the line of sight, by measuring upward from the known point is called a backsight (B. S.) or plus rod (+).

A rod reading taken on a point whose elevation is to be determined by measuring downward from the line of sight is called a foresight (F. S.) or minus rod (—).

The terms foresight and backsight do not refer to the direction in azimuth in which the telescope may be pointing, since azimuths are not considered in leveling, but rather to the rod reading. A backsight on the level rod determines the height of instrument (written H. I.) just as a backsight of a transit determines its proper orientation, and a foresight on the level rod determines the elevation of a new point just as a foresight of the transit determines the azimuth of a new point.

A single B. S. determines the H. I., and then any number of foresights may be taken to determine the elevations of new points. Any one of these newly determined points may be taken as a reference point for determining the height of the instrument by a B. S. from a new position to which the level may be moved, and thus the determinations of elevations may be carried forward over any desired line or may cover any considered area.

When the level is moved to a new station, the first thing that is required is the H. I., and this is determined by a backsight on one of the previously determined points.

When a point on the ground at the initial rod station is taken as an origin or datum point, with a reference of zero, other points will probably lie, some below and some above, this datum, giving some negative and some positive elevations with reference to this initial point. This is inconvenient, and it is best to assume an elevation for the initial station that will place the datum or zero point below any point on the ground that is to be included in the survey. This will make all determined elevations positive, and it is equivalent to supposing that every backsight is read on a rod that is extended downward by a column of earth to the assumed datum surface.

Thus in plate No. 8, Fig. 2, the line A B represents a profile cut from the surface of the ground by a vertical plane. B. M. is the initial point and L is the first position of the level. Assume an elevation for B. M. (say 800 feet), that will surely place the datum surface C D below the lowest point of the area considered. A backsight (B. S.) or plus rod on B. M. gives a rod reading of say 9.742 feet and establishes the height of instrument (H. I.) at $800 + 9.742$ feet $=$ 809.742 feet above datum. The rod is thus virtually lengthened by 800 feet so that it reaches to the assumed datum surface at E′. Foresights or minus rods may now be read at any points within range, as at 1, 2 and 3, and these readings subtracted from H. I. (809 742) will give the elevations of the points 1, 2, 3, above datum.

Having advanced so far that the rod held at point 3 and the B. M. are equidistant from the level at L, the level must be moved forward to a new position, as L′, whence a B. S. on the known point 3 (T. P. 1) determines a new H. I. from which foresights (F. S.) or minus rods may be read to determine the elevations of new points as 4, 5, and T. P. 2. The last point is used to determine a new H. I. when the level is again moved to L″, and thus the line may be continued indefinitely. Rod readings may be taken at any points, right, left, front or rear, from any instrument station, and the line followed is not necessary or even usually a straight line. When the line of rod stations is a straight line, it is not necessary or usual to set up the level on that line.

The level rod should always be held vertically. A rodman who cannot learn to hold the rod vertically should be given some other job. There is nothing so annoying to the surveyor as the sight of a rod that cuts obliquely across the field of view. The corner levels that are designed to insure the verticality of the rod, serve only to distract the attention of the rodman and to prevent his seeing the signals of the surveyor. Moreover, the rodman needs both hands and all his attention in setting the target, and he cannot handle the corner level at the same time. The rod should be placed centrally on the supporting point and held lightly with the tips of the fingers in a balanced position. It will then be vertical. A sliding rod or target that works hard and moves by jerks should be discarded or repaired. The best remedy is an application of oil, emery and energy. If these are not effective, get a new rod. At every rod station that is to be used again as a reference point the rod should be supported on a hard unyielding point. Such supporting points are required for *Bench Marks* (B. M.) and *Turning Points* (T. P.)

A Bench Mark is a point on the upper surface of some hard, durable and fixed object, the elevation of which point has been determined or assumed with reference respectively to some fixed or assumed datum. A point used as a bench mark should be at least permanent enough to remain undisturbed as long as there will be any probable need for it. Existing objects that are suitable for permanent bench marks are: An outcrop of bed-rock; a large boulder deeply imbedded in the soil; the spreading roots of a large tree; the water table or the steps of a stone building; a stone curbing, culvert, bridge pier, retaining wall, etc., or any permanent object of a similar character. In every case the exact point that has been made a bench mark must be plainly marked. On stone a channel may be chiseled around the point leaving a projecting knob of rock on which the rod may be placed; or a copper bolt may be set in a hole drilled in the rock with its top projecting slightly and hammered to form a rounded head. On a tree, trim the root off with an axe to a convex surface and blaze the trunk. As long as the tree stands there will be no change in these marks. A new growth may cover the wound, but this may be again trimmed away and the surface of the old cut will be found intact. The scar formed by the blaze is never obliterated while the tree stands. Blazed trees forming line, corner or witness marks in U. S. land surveys are protected by a law which forbids their destruction, and all property holders are jealous guardians of such marks. A blazed tree that marks or witnesses a boundary may be considered permanent during its natural life, and is a good one to select for a B. M. or other survey mark.

When existing objects suitable for bench marks are not available, a stone monument or an iron pipe filled with concrete may be sunk in the ground below the frost line.

Temporary bench marks that will last during the continuance of the survey may be made by driving a stout stake in the ground or by driving a spike in the root of a tree or by cutting down a small tree and trimming the top of the stump to a point.

The world's universal datum surface is mean sea level, and all important government surveys in all countries are referred to this datum. The U. S. Coast and Geodetic Survey, the Lake Survey, the Geological Survey, the Boundary Surveys on the Mexican and Canadian lines, the Mississippi River, Missouri River and other river surveys are all referred to mean sea level as a datum for elevations. In all of these surveys numerous permanent bench marks have been established.

Many railroads refer elevations to mean sea level, and all large cities should have bench marks referred to the same datum.

Every survey should be referred to mean sea level if possible. Therefore, before beginning a survey, apply to the Chief of Engineers U. S. A., the Superintendent U. S. C. & G. Survey, the Director of the U. S. Geological Survey, the Department of State, Treasury, War, or Interior, the chief engineer of the nearest railroad, or the city engineer of the nearest town for the location, description, and elevation of a bench mark in the desired locality. If the desired information is obtained, use the established bench mark as the basis of elevations, and thus refer the survey to mean sea level.

On every map the principal bench mark used in the survey should be shown by a small square marked B. M. No. —, and a legend on the map should give number, elevation, and full description of each B. M. thus:

"B. M. No. 1. Fort Leavenworth, corner Scott and Kearney Avenues, stone building (old Commissary, now Post Laundry) southwest corner of building; a square notch cut on water table.

"Elevation 896.723 feet above mean sea level."

In describing a bench mark give first the general locality (Fort Leavenworth), then the particular locality (corner Scott and Kearney Avenues), then the object on which the bench is marked (stone building, Post Laundry), then the place on the object (southwest corner of building), then the exact point (square notch cut on water table), then the elevation, and finally the datum service, if this is fixed, and not assumed. If assumed, it should be so stated.

A *turning point* (T. P.) is a momentary bench mark used only while the level is moved from one position to another. *The level should never be moved or disturbed at any set-up* until a convenient T. P. has been established and read by a foresight. Then the level may be moved and the new H. I. determined by a B. S. on T. P.

In foresights that are taken simply to determine the surface elevations of points on the ground the rod may rest on the natural surface, but when the foresight is taken to determine a T. P., the rod must rest on a hard fixed point that will suffer no change while the level is being moved. The rodman should therefore carry a strong iron pin with a large rounded head (Plate 8, Fig. 3). When a T. P. is required he drives the pin in the ground and uses it as a point of support for a foresight or minus rod. When the level has been advanced to a new position the rod is again held on the T. P. for a backsight or plus rod to determine the new H. I. The rodman then pulls the T. P. pin and advances to new rod stations. A ring of iron or a loop of rope attached to the pin makes a convenient handle for pulling and carrying. The T. P. pin and a hatchet should be slung to the belt of the rodman so that his hands may be free to manipulate the rod.

The rod must be placed gently on T. P. to avoid any disturbance of the pin. The bottom of the rod and the top of the pin must be wiped or brushed clean at every T. P. to make sure that there is interposed no mud, dust, sand or grass at the point of contact.

When a line of levels is run along a paved road or walk, or over stony ground, the pin need not be used for T. P's. The surface of the pavement or the top of a firm stone will serve as a T. P. Care must be taken that the rod is held on the same point for both F. S. and B. S.

It has been stated that adjacent turning points should be equidistant from the level, or that a new T. P. should be placed as far in advance of the level as the previous T. P. or B. M. is in rear of it. The two reasons for this are as follows:

First, if the instrument is not in perfect adjustment the line of collimation will not be exactly horizontal, but at equal distances from the level in all directions the departure of the collimated line of sight from the horizontal will be the same. In other words, the line of collimation rotating on the vertical axis will generate a cone instead of a plane, and circles of this cone will lie in horizontal planes. Rods equidistant from the level will pierce this cone in the same horizontal circle, and the difference of their readings will therefore give the true *difference* of elevation, which is all that the level and rod give in any case.

In Plate No. 9, Fig. 1, let L be the position of the level; m and n, two turning points equidistant from L, and suppose the line of collimation to be not truly horizontal. If it inclines upward the rods would be read at the points M' and N' respectively, both in the horizontal M' N' giving a difference of elevation, d. If it inclines downward, the rods would be read at the points M" and N" respectively, in the horizontal M" N" giving the same difference of elevation, d. If it is horizontal the rods would be read at the points M and N respectively, and the difference of elevation, d, would be the same. If, however, the first rod be held at m, and the second at p or t, at a lesser or greater distance from L, the elevations would not be measured from the same horizontal and the measured difference of elevation would not be correct.

Second, (Plate No. 9. Fig. 2) vertical measurements for determining difference of elevation should be made from a level surface M' L N' rather than from a horizontal plane M L N. The departure of the level surface from the horizontal plane is 0.001 ft. in a distance of 200 feet and varies nearly as the square of the distance. At twice 200 feet it is 0.004; and at three times 200 ft. it is 0.009 ft., etc. But at equal distances from the level the departures of the level surface from the horizontal plane are equal, and do not affect the determined *difference* of elevation, d.

Refraction acts as a partial correction of curvature because it bends the line of sight downward and brings it nearer to the level surface than is a true horizontal line. Thus the lines of sight (Plate 9, Fig. 2) are actually the curved lines L M" and L N" instead of the straight lines L M and L N.

For intermediate and side sights taken only to determine the elevation of the surface of the ground it is not necessary that the distance to the rod be the same as the distance to the turning point because extreme accuracy is not possible. The rod may rest on a tuft of grass or may sink into soft ground, and a reading that is correct to the nearest tenth of a foot will suffice, since the error affects nothing else. But if an error be made in the foresight or backsight on a T. P. it affects all subsequent elevations. The equal distances from level to turning points may be determined with sufficient accuracy by pacing.

In a clear, steady atmosphere the rod or target can be seen distinctly at a distance of 500 to 600 feet. Unfavorable weather conditions, such as haze, mists, or heat may reduce the range of distinct vision to 300 or 400 feet.

On steep slopes the length of sight is determined by the height of the telescope above the ground or by the length of the rod. (Plate 9, Fig. 3.)

In leveling up hill the level cannot be advanced beyond a point, as L, whence the top of the rod at T. P., m is visible, and the new T. P., n, must be placed below a point where the line of sight pierces the ground. In leveling down hill, the level must be so placed that the foot of the rod at T. P., n, shall be visible, and the new T. P., m, must be so placed that the top of its rod shall be visible, in the line of sight. It is difficult to determine by eye the proper place for setting up the level and the surveyor will often find after he has leveled the instrument that the line of sight passes just below the foot or just above the top of the rod. He then moves the instrument and sets it up again. This trouble may be avoided by the use of a hand level, which will show at a glance whether the trial point is too high or too low.

Differential leveling is the operation of determining, with level rod and level, the vertical distance from one point to another. When the horizontal and vertical distances between the two points are small, a single set up of the level giving a rod reading on each point is sufficient, because the algebraic sum of the two measurements will give the required vertical distance. When the horizontal or the vertical distance is great, a series of vertical measurements must be made, and the algebraic sum of all the vertical measurements will give the vertical distance from the initial point to the objective point.

Thus let it be required to measure the vertical distance from A to E (Plate 10, No. 1). If the lines A A′ and E E′ represent level surfaces through the points A and E respectively, then the vertical line A′E′ represents the required vertical distance. It is measured as follows:

PARTY.	EQUIPMENT.
Levelman......................	1 surveyor's level.
	1 hand level.
	1 note book.
	pencils, etc.
Rodman......................	1 target rod.
	1 hatchet.
	1 turning point pin.
	1 note book.
	pencil, etc.

The initial point, A, must be definitely marked. It is usually a bench mark.

The levelman sets up the level at a convenient point, as L, making sure by a trial sight through the hand level that the horizontal line of sight will not pass above the top of the rod which the rodman holds at A (or, on a descending slope, that it will not pass below the bottom of the rod). By a backsight, the plus rod A a′ is read. The rodman records the reading and moves forward counting his paces to the level at L^1. The levelman examines the rod and records his reading, and then compares his record with that of the rodman. If they agree, the reading is assumed to be correct. If not, the rodman returns to A and a new reading is taken. When the record is verified the rodman moves on, counting paces till they equal the number that brought him to L^1, and establishes a turning point at B, the distance L^1B being equal to AL^1.

By a foresight, the minus rod, b B, is read and the rodman records it.

The levelman carries the level forward and examines the rod at B. Having compared his record with that of the rodman, he selects a new position, L^2, sets up the level and by a backsight reads the plus rod B b′ on the same turning point, B.

The rodman pulls the turning point pin and again advances, and the operations just described are repeated and continued till the rod finally rests on the objective point E, where the last foresight is taken.

The measurements that have thus been made are $+ Aa', - bB, + Bb', - cC, + Cc', - dD, + Dd', - eE$, and their algebraic sum is equal to A′E′, the required vertical distance from A to E.

The record consists simply in writing down the successive rod readings and giving to each its proper sign. For convenience, it is best to write all the plus rods in one column and all the minus rods in another column. The sum of the latter subtracted from the sum of the former gives the required vertical distance, thus:

DIFFERENTIAL LEVELING.

Place........................ Date..........................
Levelman..................... Rodman........................

Rod Station.	B. S. +	F. S. —	Remarks.
B. M. 1	9.426		Elevation 623.541 feet.
T. P. 1	8.219	1.748	
T. P. 2	2.088	6.345	
T. P. 3	7.531	5.306	
B. M. 2		3.610	Elevation 633.716.
+	27,264 —	— 17.089	=Vert. dis. = + 10.175.

The exact position of the level is immaterial, provided it is equidistant from successive turning points, and no record of level stations or of horizontal distances is necessary. The level serves the same purpose in vertical measurements that the stake or pin and the plumb line serve in horizontal measurements, namely, to indicate the points from and to which the measurements are to be made.

In differential leveling over long lines the work is expedited by employing two rodmen and an axeman. While the foresight and backsight are being taken on one rod, the other rodman paces the distance to the new rod station, drives the T. P. pin, and is holding the rod in place when the levelman is ready for a new foresight.

The axeman clears away brush, weeds or other obstructions from the line of sight, or when not needed for that purpose, he carries the level forward, leaving the levelman free to verify the rod readings and to use the hand level in selecting new level stations.

When self-reading rods are used the levelman does all of the reading and recording, and the rodmen confine their attention to holding the rod vertical.

Profile Leveling is the operation of measuring, with level rod and level, the vertical distances from an assumed datum and with tape or chain in horizontal distances from an assumed origin, to points on the ground along a given line, in order to determine a profile of that line.

Thus in (Plate X, Fig. 2,) let A B represent the datum plane from which vertical distances are to be measured, and the point "o" be the origin from which horizontal distances are to be measured, and let B. M. be a bench mark of known or assumed elevation.

The points selected for determination are generally taken at equal intervals of 100 feet along the line, but in addition to these, all points where there is a change of slope must also be determined. The 100 ft. stations are numbered consecutively 0, 1, 2, 3, etc., for 0, 100, 200, etc., feet from the origin. Intermediate critical points are called *plus stations*, and are designated by the number of feet from the preceding station; thus 2 + 52 and 6 + 32 are stations 252 and 632 feet respectively from the origin.

The principles involved have already been explained. The method is as follows:

PARTY.	EQUIPMENT.
One Levelman	Surveyor's level, hand level, note book, pencils, etc.
One Rodman	Level rod, turning point pin, hatchet, note book, pencils.
Two Chainmen	Chain or tape (100 ft.), marking pins, plumb lines.
One Axeman	Axe, stakes and marking chalk or crayon.

If two rodmen are employed the work will be expedited.

In an engineering survey made for the purpose of laying out a road, railroad or other structure, stakes are driven at every station and are marked with the number of the station in order that located points may be identified and used during construction. In topographical surveying the rod is simply held at the marking pin used in chaining. When it has served this temporary purpose the pin is pulled and retained by the chainman for further use. When a "plus-station" is needed at a critical point, the chain remains in place until the distance to the rod at the "plus-station" is read and then the chain is carried forward to locate the next rod station. In this way the chaining and leveling proceed together, and the rod stations are not permanently marked on the ground.

The level is set up at a convenient point, as L, (Plate 10, Fig. 2) whence sights on the B. M., and on the initial point, and on points along the line can be secured. A backsight on the B. M. gives the H. I. The chain is stretched horizontally from 0 to 1, 1 to 2, 2 to 3, etc., along the line of the desired profile, and these points are marked momentarily by marking pins while the chain is carried forward and a rod read at the marked point. Between 2 and 3, and between 3 and 4, in the case shown in Plate 10, Fig. 2, plus-stations are required because the slope changes, and in each case the chain is held in place until the positions of the rod at 2 + 52 and 3 + 46 respectively are determined. At Sta. 4 the distance of the rod from the level is about equal to the distance of the level from the B. M., and a turning point is required.

Foresights having been read on 0, 1, 2, 2 + 52, 3, 3 + 46 and T. P. at 4, the level is moved forward to L_2 and a new H. I. is determined by a backsight on the T. P. These operations of chaining and leveling, that is, of measuring horizontal and vertical distances, are continued to the end of the line.

The level should not be set up *on* the line because there it may interfere with a rod station or be so close to a rod station that the rod cannot be read. If it is placed 15 or 20 feet from the line on either side there will be no danger of interference.

Profile leveling involves three simultaneous operations, as follows:

1. A line of differential levels, which includes the readings on B. M. and T. P's only.

2. Reading foresights from each instrument position to the selected rod stations along the line.

3. Chaining, to determine the horizontal distances from the origin to the successive rod stations.

These operations determine the rectangular coördinates, in the vertical plane, of the points considered. When the direction of the line is also known, the three coördinates which fully determine each point with reference to the origin are known.

THE RECORD.

The following form of record is the one generally used:

Survey of ....
(Place) .... (Date)..........................
Levelman .... Rodman ....

1	2	3	4	5	6	7	8
Sta.	B. S. (+)	H. I.	F. S. (—)	El.	Elev. Grade	Cut— Fill +	Remarks.
B. M.—	10.463	282.326		271.863			
0			9.5	272.8			
1			7.6	274.7			
2			5.4	276.9			
+ 52			4.9	277.4			
3			5.1	277.2			
+ 46			5.5	276.8			
4 T. P_1			7.302	275.024			
4 T. $P._1$	4.062	279.086		275.024			
5			4.8	274.3			
6			7.0	272.1			
+ 32			7.9	271.2			
7			6.6	272.5			
T. $P._2$			5.846	273.240			
T. $P._2$	9.806	283.046		273.240			
8			8.9	274.1			
9			4.2	278.8			
+ 85			2.8	280.2			
10			3.1	279.9			
+ 70			4.0	279.0			
11			6.7	276.3			
T. $P._3$			7.185	275.861			
Etc.,				Etc.			

The first column is the record of horizontal measurements by chaining.

The entries after B. M. and T. P. stations are the record of the differential leveling by means of which accurate control of vertical distances is maintained.

The fourth and fifth columns are respectively the vertical measurements and the computed vertical distances giving the vertical coördinates of the required points.

The known elevation of the B. M. is 271.863. A B. S. on this B. M. of + 10.463 gives an H. I. of 271.863 + 10.463 = 282.326. From this H I. the foresights, —9.5, —7.6, —5.4, etc., on stations 0, 1, 2, etc., respectively, are read and subtracted from the H. I. to give the elevations, 272 8, 274.7, 276.9, etc., of these stations respectively. The station rods are read only to the nearest tenth of a foot. Station 4 is also a T. P. and therefore its foresight (—7.302) is read to the nearest thousandth, giving an elevation of 275.024.

The level being moved and set up at a new position, L_2, the new H. I. is required and is determined by the B. S., + 4.062 on the T. P., giving H. I.= 275.024 + 4.062 = 279.086. From this H. I. the foresights, — 4.8, — 7.0, — 7.9, etc., are read on stations 5, 6, 6 + 32, etc., giving the corresponding elevations 274.3, 272.1, 271.2, etc. The next T. P. does not fall at a station and is not necessarily on the profile line. As a rule it is better not to use the same point as station point and T. P. because the T. P. pin is not always driven flush with the surface of the ground.

The record is carried forward in this manner, as the work proceeds, to the end of the line. The total distance covered may be only a few hundred yards in a small topographical survey, or it may be tens or hundreds of miles in a road or railroad survey. The line along which the profile is run is not necessarily or usually a straight line, but if not straight it is made up of straight portions.

It is not always convenient to begin a line of levels near the B. M. which is to be used as a basis of elevations for the survey. For instance, a line of levels is to run from X to Y (Plate XI, Fig. 1) and the B. M. to which elevations must be referred is at K.

The level is set up at the points L_1, L_2, etc., in succession, and rods are read on the stations 1, 2, 3, etc., and on the T. P's 1, 2, 3, etc., as already described, and no elevations are known and no H. I. can be determined until the position L_8 is occupied, whence a rod may be read on the B. M. at K. Then working backwards through the notes, and reversing the signs of the backsights and foresights on the previous T. P's, the required H. I's and elevations are computed. The foresights on the previous station points 0, 1, 2, etc., remain negative unless the station point is also a T. P.

On long lines of levels, whether differential or profile, bench marks should be established at frequent intervals, not more than a mile apart and preferably half a mile. Existing permanent objects that are suitable for bench marks should be utilized when possible, and every bench mark should be plainly marked and fully and accurately described. It should always be borne in mind that a survey is made for future use, and that years may pass before its purpose is accomplished. The records and maps of a survey acquire increased value with age, so long as there remain points on the ground that can be identified in the records or on the map of the survey. Therefore, points that give the greatest promise of permanency should be the ones most accurately located on the map and most carefully and fully described in the records.

In the form of record given for profile leveling the columns headed "Elevation of Grade" and "Cut or Fill" are used only in engineering surveys and when the ground is to be graded for a road, railroad, canal, ditch, reservoir or other structure. When the grade line or surface has been established its elevation at each station point is determined and entered in the column "Elevation of Grade." The elevation of the ground at each station point subtracted from the elevation of grade at the same station gives the "Cut or Fill" at that station; a negative result indicating *cut* and a positive result indicating *fill*. These results are entered with proper signs in the column headed "Cut or Fill," and the same results with proper signs or with the words "cut or fill" are marked on the corresponding station stakes. For narrow ditches or for trenches in which a pipe line is to be laid the marking of the "cut" on the line of grade stakes is all that is necessary, but in laying out the work for the construction of roads, railroads, canals, etc., where the surface of the ground is to be permanently and extensively modified by excavations and embankments the profiles cut by the oblique side planes that will limit the excavations and embankments must also be marked on the ground by stakes, and the volumes of excavation and embankment must be computed. This is not properly a part of topographic surveying, but is rather an application of its methods and principles to a very useful purpose.

Plotting the Profile.—This consists in laying off on paper to a convenient scale the horizontal and vertical coördinates determined respectively by chaining (column 1 of record) and by leveling (column 5), and then drawing a continuous line through the plotted points. The line so drawn represents, in vertical projection, the profile of the ground on the line which has been surveyed.

Thus (Plate 11, Fig. 2) draw the line A B, assume the origin at the point "0," and lay off with a convenient scale the horizontal distances given in column 1 of the record. This locates on the plot the station points 0, 1, 2 + 52, 3, 3 + 46, 4, etc., at which the vertical measurements were made. At these points erect perpendiculars, and on each perpendicular lay off with a convenient scale the elevation of corresponding point. Mark the points thus determined. The line 0′, 1′ 2′, 3′, etc., drawn through these points is the plotted profile.

The differences of elevation of points on the ground are usually small as compared with the horizontal distances between the same points. A scale suitable for laying off horizontal distances is therefore too small for vertical distances, and the vertical scale should be exaggerated in order that the differences of elevation may be plainly appreciable. In the example given in Plate 11, Fig. 2, the horizontal scale is 1 in. = 200 feet, and the vertical scale is 1 in. = 10 feet. Note also that the line A B does not represent the datum or zero surface, but is given an elevation of 270 feet. It would be merely a waste of paper and time to lay off the whole vertical distance from the datum plane up to the points considered. Therefore only that part of the vertical scale whose readings are required (270 to 281) is constructed, as shown at A C (Plate 11, Fig. 2).

To facilitate the plotting of profiles *profile paper* is used. It is ruled, by hand or by machine, with horizontal and vertical lines that are equally spaced in each direction. When great accuracy is required the paper should be ruled by hand, but for ordinary purposes the machine ruled paper, furnished by dealers in drawing supplies, will suffice. Standard profile paper has $\frac{1}{4}$-inch spaces for the horizontal scale and $\frac{1}{20}$-inch spaces for the vertical scale. Every tenth line in both directions is drawn heavy, and the heavy lines are therefore 2.5 inches apart horizontally and $\frac{1}{2}$ inch apart vertically. These divisions are suitable for horizontal scales of 1 inch = 100, 200, 400, 500, 600, 800, 1000 feet, etc., and for vertical scales of 1 inch = 10, 20, 40, 50, 60, 80, 100 feet, etc., the vertical scale being any convenient multiple of the horizontal scale.

On profile paper the ruled lines are scales for laying off horizontal and vertical distances on any part of the drawing, and no other scale is needed. Points that have been determined by rectangular coördinates can be plotted quickly and accurately, and then the profile lines determined by these points may be drawn. When a profile line is thus drawn the elevation of any point on that line may be scaled off at once.

When the horizontal and vertical distances are more nearly equal, the same scale may be used in both directions, and in this case the paper is ruled in squares and is called *cross-section* paper. Standard cross-section paper is ruled ten lines to the inch in both directions. This paper is very convenient for drawing cross-sections of excavations, embankments, terraces, etc., and for plotting any rectangular coördinates to the same scale in both directions. It may also be used as profile paper.

To plot a profile in plan, or horizontal projection, the directions and length of the straight portions of the line must be known, and these are plotted like any traverse. The rod stations are then laid off along the traverse line, and each is marked with its determined elevation. Elevations of intermediate points may be found by interpolation.

Cross-section leveling or cross-sectioning is the operation of running profiles on lines that are perpendicular to a base profile and that pass through the station points of the base profile. Any convenient rod stations of the base profile may be used as bench marks from which to run the cross profiles. Several cross profiles may often be run from the same instrument stations.

Beginning at the bench mark, B. M. (Plate 12) run the base profile 0, 7, and mark each rod station with stake, number and elevation. At each rod station lay off and range out a perpendicular, and on the perpendiculars measure off the 100 ft. stations and mark them with stakes numbered 0 R 1, 6 R 2, etc., or 3 L 1, 5 L 2, etc., according as they are to the right or left of the base profile. A point, such as A, that does not lie on a line may be designated by the plus distances from the previous station, thus: 3 + 45 R 2 + 60.

When the stations have been properly marked, set up the instrument at a convenient point, as I_1, whence a backsight on the B. M. and foresights on several adjacent stations may be read. Having completed these readings, set a T. P. at 1 R 3 and move the level to I_2. Read B. S. on 1 R 3 and foresights on accessible stations, and so proceed to the right as far as the cross profiles extend. If they end at R_5, move back to I_3, and using station 2 of the base profile as a T. P. to determine the H. I., read rods on adjacent stations and then move to I_4, etc. In Plate 12, the instrument stations I_1, I_2, I_3, etc., are marked thus O, the turning points are marked thus □. The backsights are denoted by full lines, and the foresights by broken lines.

Note that the stations of the base profile are used as turning points whenever they are within reach of the instrument station, on the principle that all measurements should refer back by the shortest and most direct line to the origin. The base profile is run for the purpose of providing check points or starting points from which the cross profiles may be run.

On broken ground, where the slopes are steep and varied it would be necessary to use many more instrument stations than those that are indicated in Plate 12, and many plus-stations to determine critical points would be required. In leveling, the labor required increases directly with the steepness of the slopes, and also with the frequency of the obstructions that interrupt the view.

Plate 12 is only a diagrammatic illustration of a possible case, and it must not be taken as a guide for general application. The lay of the ground and the judgment of the surveyor must determine in each case the course to be pursued and the positions to be selected for instrument stations and turning points.

The record and the reductions are the same as those for profile leveling.

In locating or in laying out a road, railroad or canal, a long narrow strip of country, following the proposed route, is surveyed, and the *leveling* consists in running a base profile along the middle line of the proposed work with cross profiles of only one or two stations on each side, at every 100 ft. station of the base profile. In this case the cross profiles can usually be run at the same time with the base profile and from the same instrument stations. On rough ground the necessary "plus stations" at critical points must also be established.

Cross-sectioning with chain or tape and level rod is an application of the rectangular system of co-ordinates as a method of surveying. It is used principally in engineering surveys, but may be used for topographical surveys of small areas.

To plot a cross-section survey lay off the paper in penciled squares to a suitable scale and mark the corners of the squares, with numbers arranged like those that designate the stations on the ground. Mark each station point also with its determined elevation. Interpolate contour points on the lines of the squares and draw contour lines, properly joining the points that have the same elevation. Plot other features and incidents of the surface, such as roads, streams, gullies, woods, buildings, etc. (the notes should include the measurements necessary for this purpose). After inking the map, the penciled squares and the elevations of stations may be erased. The result should be an accurate topographical map of the area covered. This method of surveying is too laborious to be used for large areas.

CONTOUR SURVEYING.

The term, "contour surveying," is here applied to a method of surveying in which contours are located or "run out" on the ground by means of a level and level rod, and points on the contours are located by means of the transit and stadia. Both operations may be performed with a transit which has a level tube attached to the telescope.

The two separate operations are carried on at the same time, viz: a stadia survey, running traverses and locating all necessary points by side shots, and a level survey, establishing turning points and running contours.

This method is best described in connection with the record, which is kept as follows:

RECORD.

Place.................................... Date....................................

Contour survey of..

Observer..

Traverse Notes.							Level Notes.					
At Sta.	Point Obs'd.	Azimuth.	Bearing.	Stad.	Vert. Ang.	Hor. Dist.	B. S. +	H. I.	F. S. —	Elev.	Set Rod.	Rem.
1	B. M. 2	182–40	S 2½ W	122	0	122	6.316	886.690		880.374	1.690	
	885	225–00		451								
	885	216–00		394								
	T. P. 1								2.000	884.690		
	Sta. 2	230–15	S 50 W	420	—0–13	420						
2	Sta. 1	50–15	N 50½ E									
	T. P. 1						5.903	890.593		884.690	5.593	
	885	170–00		120								
(Etc., until 885 is run out. Then to place Sta. for 880.)												
	T. P. 2								9.725	880.868		
	Sta. 3	220–00	S 40 W	224	—5–30	222						
3	Sta. 2	40–00	N 40 E									
	T. P. 2						2.000	882.868		880.868	2.868	
	880	182–40		150								
	880		Etc.					Etc.				

The party should consist of observer, level rodman, stadia rodman and axeman. Set up the transit at the initial station (1) and orient by the compass or by sighting at a triangulation station whose azimuth is known. Level the telescope by its level tube and backsight (+6.316) on the B. M. whose known elevation is 880.374, finding H. I.=886.690. If contours are to be run at five feet V. I., the contour next below this H. I. will be 885, and the target will be set on the rod at 1.690. The stadia rod is now placed at B. M. and and the observer reads azimuth, bearing and stadia distance to locate the B. M. The level rod is then carried up or down the slope as signaled by the observer until, when resting on the ground, its target is bisected by the horizontal wire of the telescope. Since the H. I. is 886.690, and the target is set at 1.690, the foot of the rod will rest at a point of the 885 contour. The stadia rod is placed at this point and its azimuth (225–00) and stadia distance (451) are read and recorded. Other points on the 885 contour are in like manner found by placing the level rod and located by readings on the stadia rod.

When a new station is required a T. P. is established by a F. S. (—2.000) giving its elevation 884.690, and the selected new station (2) is located by azimuth 230–15, stadia 420 and vertical angle —0.13. This angle is so small that no reduction to the horizontal is necessary, and the horizontal distance is 420.

Setting up at station 2, the transit is oriented by a backsight (50–15) on Sta. 1, and the new H. I. is determined by a backsight (+5.903) on T. P. 1, giving H. I. =890.593 and a new setting of the rod (5.593) for the 885 contour.

When 885 has been run out to the limits of the survey, a new station and T. P. are selected and located for beginning the next contour, say 880. A F. S. (—9.725) gives elevation 880.868 for T. P. 2, and azimuth 220–00, stadia 224, vert. ang. —5.30 (giving horizontal distance 222), locates station 3.

At station 3, a backsight 40–00 on Sta. 2, orients the transit and a backsight (+ 2.000) on T. P. 2 gives H. I. 882.868, and a rod setting of 2.868 for the 880 contour.

If the instrument stations be properly selected with reference to H. I., two contours may be run at the same time by using two targets or marks on the level rod. Thus, in the foregoing example, at station 1 set one target at 1.690 and another at 6.690. The first will locate points on the 885 contour, and the second, or upper target, will locate points on the 880 contour.

Vertical angles are read on traverse courses only, and are used to reduce stadia readings to the horizontal. Elevations are carried along by the level observations. The telescope must be re-leveled for each sight on the level rod.

If a rod be made to show stadia readings on the face and level rod readings on the back, only one rodman will be required. He turns the level rod markings toward the observer for the level readings, and the stadia face toward the observer for stadia readings.

On a long straight stretch of contour only a few points need be determined, but on broken ground, where the contours curve irregularly, numerous points are required, and especially those at salients and reëntrants.

In plotting this survey the traverses are first adjusted and plotted by methods previously described. Then side shots are plotted and contours are drawn through the points having the same elevations. No interpolation of the contour points is required in this case. The notes should, of course, include readings necessary to locate roads, buildings, streams, etc., and these are plotted and drawn in the usual manner.

PLATE VIII

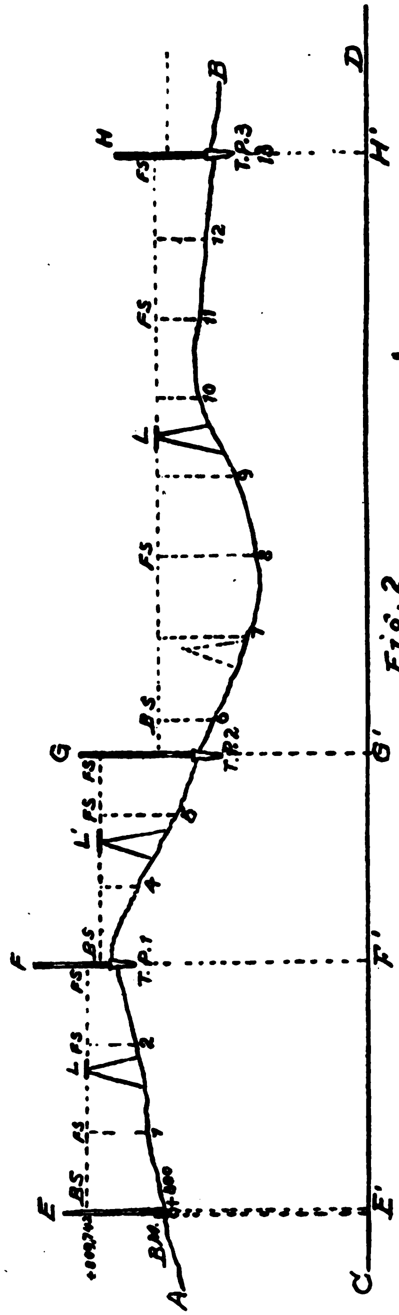


Fig. 2

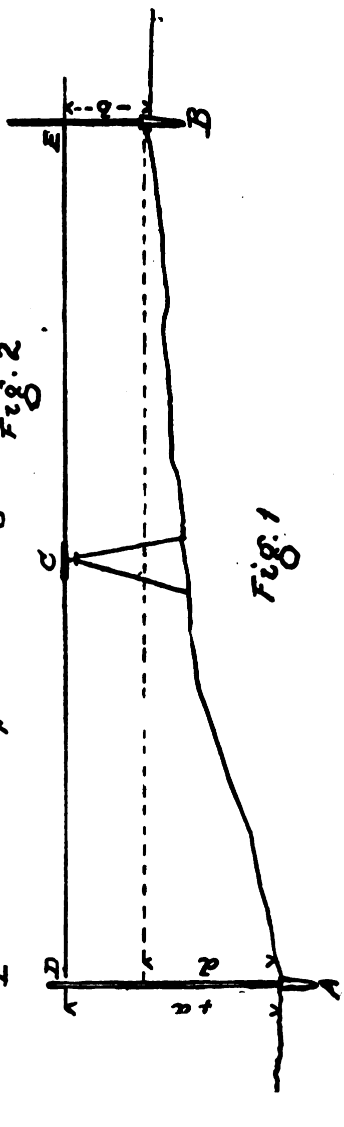


Fig. 1

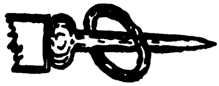


Fig. 3

PLATE IX

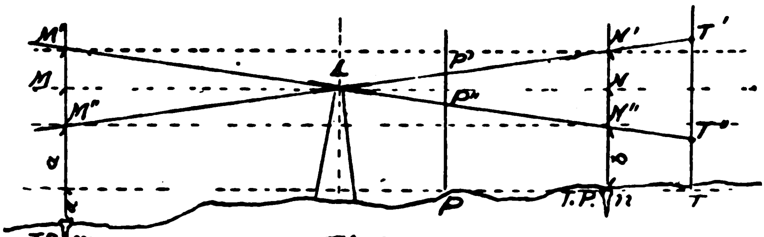


Fig. 1

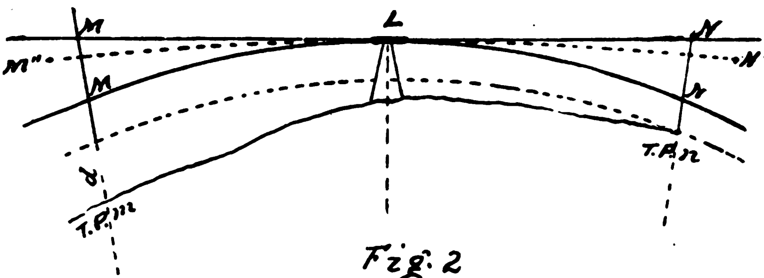


Fig. 2

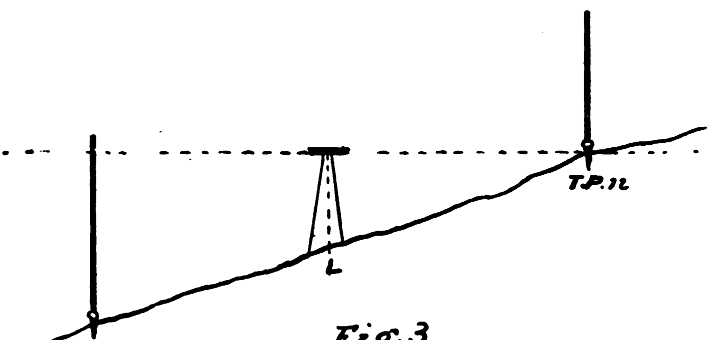


Fig. 3

PLATE X.

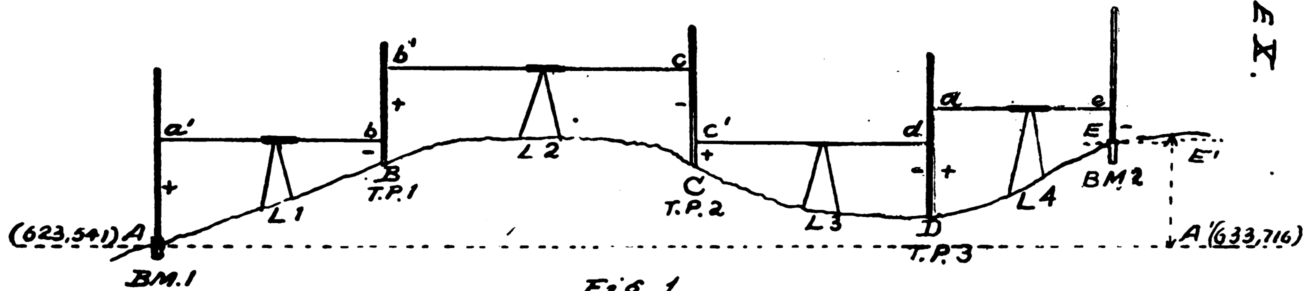


Fig 1

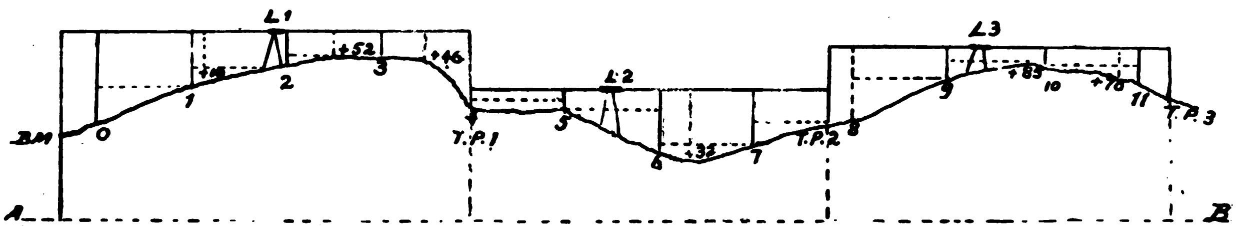


Fig 2

PLATE XI

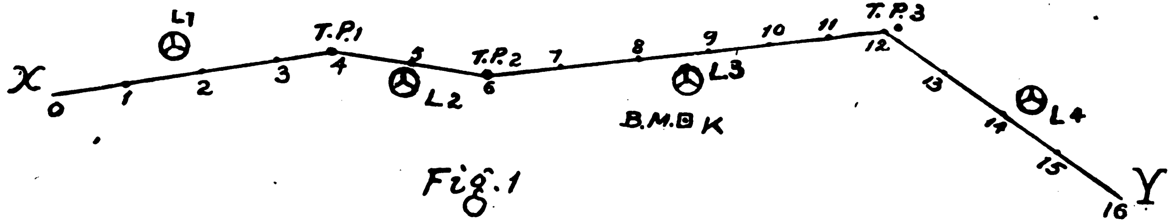


Fig. 1

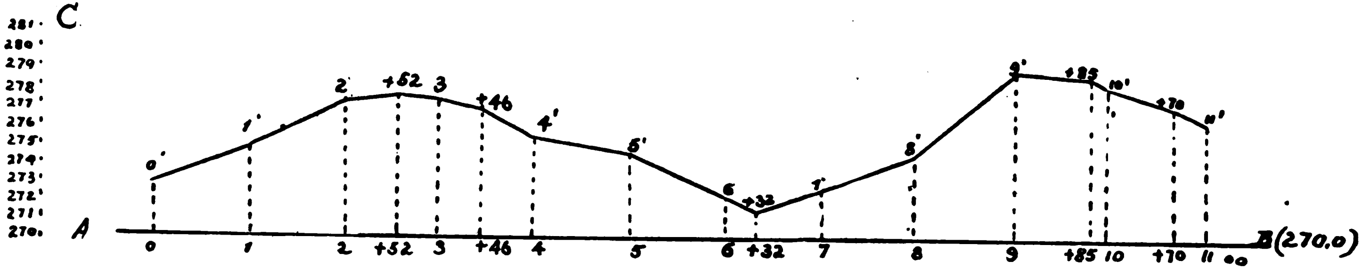
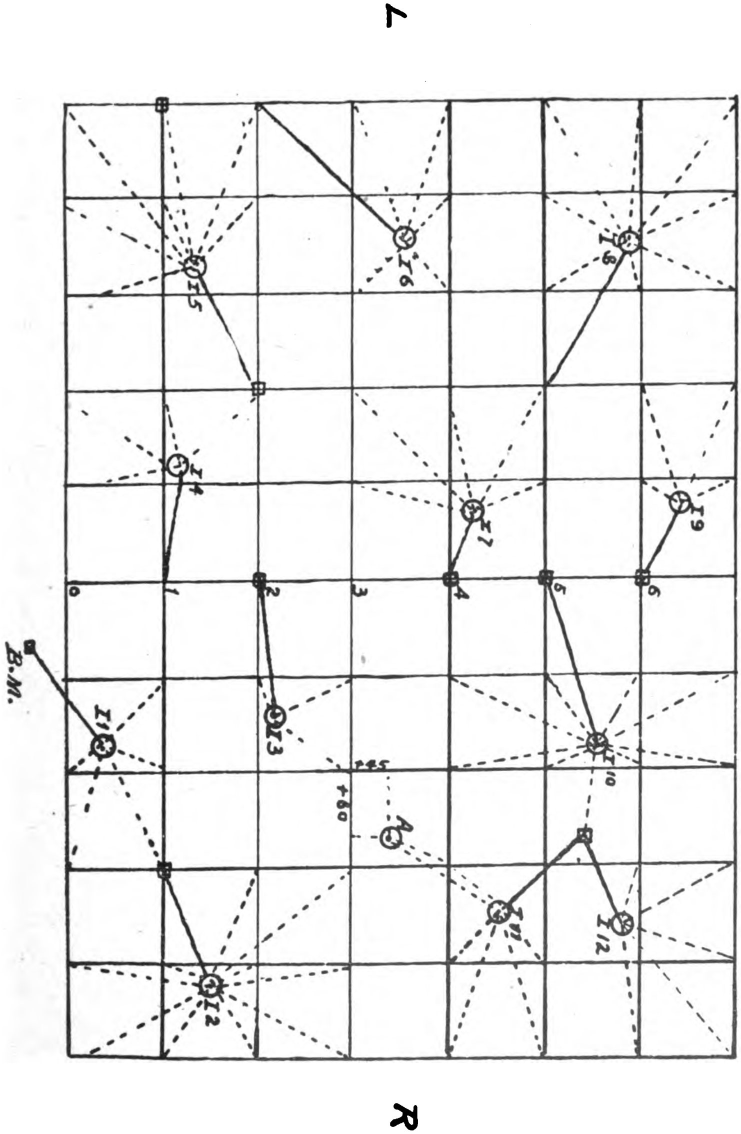


Fig 2

PLATE XII.



CHAPTER XII.

THE CHAIN.

The chain is made of short pieces of steel wire joined end to end by small rings of the same material. A link is the distance included between the middle points of adjacent joints. The handles at the ends of the chain are included in the end links. The length of the chain is the distance from outside to outside of the handles when the chain is stretched taut.

The Gunter's or Surveyor's Chain has 100 links, each 7.92 inches long. One chain = 4 rods = 22 yards = 66 feet = 100 links = 792 inches.

One mile = 8 furlongs = 80 chains.

One acre = 10 square chains.

One section = 1 square mile = 640 acres = 6,400 square chains.

The Surveyor's Chain is convenient for measuring land in order to determine the acreage. The links are recorded as hundredths of a chain; thus, 8 chains and 35 links is written 8.35 chains. When the area is computed in square chains, the acreage is found by moving the decimal point one place to the left. Thus a rectangle 8.35 chains wide and 10 chains long contains 83.5 square chains or 8.35 acres.

Ten chains = 1 furlong = 1/8 mile, therefore the surveyor's chain is convenient in laying out a race track where the quarters and eighths of a mile are important units.

The Engineer's Chain has 100 links, each 12 inches long, and it is therefore a scale of feet, 100 feet long.

Half chains of fifty links are also furnished by the makers. These are 33 feet long for the surveyor's half chain and 50 feet long for the engineer's half chain.

The metric chain has 100 links each 0.2 of a meter or 7.874 inches long. It is therefore 20 meters in length and has 5 links to the meter. It is easily mistaken for a Gunter's chain, as it is only 4.6 inches shorter.

Chains are usually compared with a standard and adjusted to standard length by the makers, who will furnish also a statement of the pull or tension required to stretch the chain to standard length at a stated temperature. New chains may therefore be depended upon to give fairly accurate measurements of distance. With use, the rings and loops wear away at the points of contact, and may become elongated under tension, or may be partly opened if they are not brazed together. These causes tend to lengthen the chain, and an adjustment to standard length becomes necessary. This is done, if a standard is available, by comparing the chain with the standard and adjusting one of the handles by means of its adjusting screw and lock nuts so that the chain under normal tension and temperature shall be equal to the standard in length.

When a chain is used to measure a line with the greatest attainable accuracy a correction for temperature must be applied. This correction is the same for the chain as for the steel tape and will be explained in connection with the latter.

Steel pins are used to mark the successive chain lengths on the ground. They are called "marking pins," and are about 14 inches long, pointed at one end and bent into a loop at the other in order that they may be strung on a ring of strong wire like keys on a key-ring. Eleven pins form a set. One pin marks the origin or initial point of the line to be measured. When the other ten pins have been used, a record of ten chains is entered in the note book, and the eleventh pin becomes the initial point for further measurements on the same line. Two pins of the set should be weighted each with a bob of lead near the point. These pins may be attached to a string to provide plumb lines for use when chaining over sloping ground. The chain should be stretched horizontally, and if the ground is uneven one end, or perhaps both ends, of the chain will be held above the ground. A plumb line is then needed to place the rear end of the chain over the last marking pin or to set a new pin under the forward end of the chain.

To *fold the chain* begin at the middle and fold the links together pair by pair, holding the bunch of links in one hand and folding with the other. Bind with strap or cord.

To *open the chain* remove the strap or cord and cast the links forward along the line while retaining both handles in the grasp.

TO MEASURE A LINE.

PARTY	EQUIPMENT
Rear chainman, - -	Chain, range pole, one weighted pin, note book, etc.
Fore chainman, - -	Ten marking pins, range pole, hatchet.

Mark the initial point with a pin, or better, with a range pole, and mark the distant objective point with a range pole if it is not already marked by some definite selected object.

The fore chainman takes one handle of the chain and moves off along the line till the chain is stretched taut. He then faces to the right, kneels on the left knee and braces the right arm against the inside of the right knee. He holds a pin and the handle of the chain in the right hand and moves to the right or left as directed by the rear chainman till the pin is on line. At the signal "Down" he forces the pin vertically into the ground with the left hand and answers "Down." He then proceeds along the line dragging the chain and clearing away brush, weeds or other obstructions that would foul the chain. He halts at the signal of the rear chainman and marks the end of the second chain in the same manner as the first. When the pin is set the forward sides of pin and handle must be even. The fore chainman can place himself nearly on line by sighting back over the last pin at the rear range pole, but the accurate alignment is maintained by the rear chainman who sights toward the forward range pole and lines in each new pin. Rags of red or white cloth tied to the pins make them clearly visible.

The rear chainman directs the work. When the chain is first stretched along the line he examines its whole length to see that there is no kink or twist at any point. At the initial point he holds the end of the chain (outside of handle) against the front of the pole or over the center mark on the stake, lines in the pin in the hand of the fore chainman by sighting at the forward range pole, calls out "Down" when the alignment is correct, and upon hearing the answer, moves forward carrying the rear handle. When he reaches the first pin he halts the fore chairman, holds his handle against the front side of the pin, lines in the next pin, and then pulls the first pin and again advances.

When the fore chainman has used his last pin there is one pin in the ground and ten in the hands of the rear chainman, and a distance of ten chains has been covered. This distance is recorded in the note book or clicked on a tally register, and the ten pins are handed to the fore chainman.

The chaining is then continued, starting at the pin that remained in the ground.

When the objective point is reached the fore chainman halts and holds the end of the chain at this point. The rear chainman lays down the chain, counts and records the number of pins that he has pulled, advances to the pin last set, stretches the chain, and reads the number of links between the pin and the objective point, which number is recorded as hundredths of a chain.

The number of record marks in the note book gives tens of chains. The number of pins in hand when the objective point is reached gives units. The number of tags between the last pin and the objective point gives tenths of a chain, and the number of links between the last pin and the nearest forward tag gives hundredths of a chain. A part of a link next to the pin may be estimated to the nearest tenth and recorded as thousandths of a chain.

For example,

Six record marks in the note book,

Four pins in hand,

Seven tags between last pin and objective point.

Three whole links between pin and next tag.

Five-tenths of a link next to the pin,
gives a total distance of 64.735 chains if it is a Gunter's chain, or 6473.5 feet if it is an engineer's chain, or 1294.70 meters if it is a 20-meter chain.

The Gunter's chain and the engineer's chain give decimal readings, while the readings of the 20 meter chain must be multiplied by twenty to give the distance in meters. A more convenient metric chain would be a chain 10 meters long with fifty links, each two decimeters in length, and with a tag at every fifth link. Each 10-chain record would be 100 meters, each pulled pin would be tens of meters, each tag would be one meter, each half-link would be one decimeter, and each tenth of a half-link would be one centimeter, making the decimal system complete.

In chaining up-hill the rear chainman must raise his end of the chain above the ground to make the chain horizontal and hold it in a vertical line through the marking pin. To do this he uses the plumb line with the weighted pin for a bob, and brings the point of the bob against the marking pin while holding the handle against the line at the proper height.

In chaining down hill the fore chainman must in like manner raise his end of the chain to the horizontal and with the plumb line find the point at which to set the marking pin. Otherwise, he may while stretching the chain, hold the weighted pin under and against the handle, and when aligned drop the pin. He pulls the weighted pin, sticks another in the same hole, and advances to the next point.

By using a staff to support the hand holding the raised end of the chain greater steadiness and accuracy are secured.

On steep slopes the end of the chain cannot be raised high enough to make the chain horizontal. In this case parts of the chain (halves or quarters) may be used in succession to keep the raised point within reach. Otherwise, stretch the chain on the ground and measure the slope with a clinometer or a slope board. Record each chain so measured, not as one chain, but as links, by a number equal to 100 × cosine of slope. This reduces the inclined measurement to horizontal distance. In practice, having measured the angle of slope of the chain length, take the cosine of this angle from a table and record it with the decimal point moved two places to the right. (Plate 13, Fig. 1.)

When part of the chain is thus used, the number of links in the partial chain must be multiplied by the cosine of the slope, and the result entered in the record.

The sum of the recorded numbers will be the total horizontal distance in links. With the engineer's chain the distance will be given in feet.

CHAINING WITH AN INCORRECT CHAIN.

A chain, rod, wire or tape of any length may be used to measure a line if the length of the chain, or rod, etc., be known. If a chain has been elongated by use, or if it has been shortened by the loss of some of the links, it may still be used as a measuring unit, if its new length be ascertained. It is only necessary to multiply the number of chains applied in any measurement by the length of the chain. The result is the required distance.

Thus, if an engineer's chain has worn at the joints so that its length, as ascertained by comparison with a standard, is 100.065 feet, a record of 6.834 chains gives a distance of $6.834 \times 100.065 = 683\ 844$ feet.

But if some of the links, say four, have been lost, and the remaining links, 96, are correct, a record of 6 chains and 83.4 links, gives a distance of 6×96 links $+ 83.4$ links $= 659.4$ links.

It is often required to lay off on the ground from a given point a given distance, and thus fix a required point. With a correct chain this is readily done since it is only necessary to apply the chain and links along the ground in the given direction the required number of times. But with an incorrect chain such application would not give the required point correctly. To lay off the given distance, divide that distance by the actual length of the chain and lay off the resulting number of chains and links.

Thus, let it be required to lay off 683.844 feet with an engineer's chain that is known to be 100.065 feet long. $683.844 \div 100.065 = 6.834$ of these chains. Therefore lay off 6 chains and 83.4 links. The point thus determined will be 683.844 feet from the given point as required.

LINING IN.

To line in is to cause a pin, pole, stake or other mark to be set in the ground on a certain line. The line may be determined by the pointing of an instrument, such as a transit, compass, sextant, or alidade of a plane table, or it may be determined by sighting through two points that have been set or selected on the ground.

When an instrument is used the observer signals to the rodman to move the rod to a point that is bisected by the vertical wire or sight of the instrument. If the required point is also to be fixed in elevation, the rodman is signaled to fix a mark or target on the rod at a point that is bisected by the horizontal wire or sight of the instrument.

When the line is determined by two points that have been marked by stakes or poles, a third point may be lined in as follows:

First. When the two given points are mutually visible:

(*a*) To line in a third point *between* the two given points, the surveyor sights from one marked point to the other and signals an assistant to set a mark in the line of sight so determined. If the surveyor is alone he sights from one marked point through the other and selects a third point that is in this line of sight; then moves forward to the desired point and sets a stake in line with either mark and the selected point.

(*b*) To line in a third point *beyond* the two marked points, the surveyor places himself on line at the desired point by sighting through the two marks, and sets a stake in the line of sight so determined.

When a third mark has been lined in, any two of the three marks may be used to fix a fourth point, and thus a line may be prolonged indefinitely by marking new points in line with any two points previously marked.

The greater the distance between the two marked points that determine a line, the more accurate will be the lining in of a new point on that line. Therefore in prolonging a line, the first mark or origin of the line should be used as one sighted point whenever it is visible from the newly determined point. In prolonging a line over undulating or wooded land the initial point is soon lost to sight and generally only the last two points will remain visible. In surmounting hills or crossing valleys, the range poles that determine the line, and the stakes that mark the line, must be placed at short intervals to be visible from the new point, but on open ground of uniform slope three marked points will keep the chainmen always on line, no matter what the included distance.

In prolonging and measuring a line in a known direction two classes of obstacles may be encountered. viz: those that obstruct the view, and those that obstruct progress along the line.

When the view is obstructed by an obstacle that cannot be cleared away, the line must be carried around the obstacle by offsets and then continued as before.

When progress along the line is obstructed by a pond, a marsh, or by private property to which access is denied, the line may still be prolonged by sighting across the obstacle, but the chaining must be carried around the obstacle by offsets, or across the obstacle by triangulation.

Many methods may be devised for passing either class of obstacle. The following simple cases are given as examples:

1. To pass an obstacle that obstructs the view, such as a building or a thicket or tall growing crops. (Plate 13, Fig. 2.) Line in and chain to C. At A and C,

on the line, 200 or 300 feet apart, lay off perpendicular offsets of equal length, A B and C D, and prolong the line B D, parallel to A C, to some point as G beyond the obstacle. Chain D E. At E and G lay off perpendicular offsets, E F and G H, equal in length to A B and C D. The line F H will be in prolongation of A C, and the lining in and chaining may be continued from F.

Otherwise, thus (Plate 13, Fig. 3): Having lined in and chained to B, construct on A B (one chain long) the equilateral triangle A B C, and prolong C B to E. Chain B E. On D E construct the equilateral triangle D E F and prolong E F to G, making E G equal B E. On H G construct the equilateral triangle H G K. The side K G will be in prolongation of A B, and the triangle B E G will be equilateral. The distance B G is therefore known because it is equal to B E or E G. The line K G may now be prolonged and chained.

2. To pass an obstacle that obstructs progress along the line, but which does not obstruct the view. (Plate 13, Fig. 4.) Prolong the line by sighting across the obstacle and lining in C E. Having chained to A, lay off perpendicular offsets A B and C D of equal length, and chain B D. Then continue chaining on the line C E.

If the obstacle is a stream too wide to be spanned by the chain it may be passed by the method indicated in Plate 14, Fig. 1. Make the angles at A and B each equal to 60° by constructing equilateral triangles, and chain A B. Find the point C beyond the stream where A D and B E prolonged intersect. Then A C = A B, and the chaining may be continued from C.

Otherwise, (Plate 14, Fig. 2) construct the 3-4-5 triangle A B C, right angled at A, and at any point D, beyond the stream erect a perpendicular and find the point E where it interests C B prolonged. Measure D E. Then D E: D B : : A C: A B : : 3 : 4. Hence D B $= \frac{4}{3}$ D E.

Second. When the two given points are not visible each from the other.

In this case the line joining the two points cannot be determined by sighting direct from one point to the other, and the surveyor cannot begin at one point and chain toward the other because he cannot see the other. He must first determine and mark on the ground, by marks that are mutually visible, the line that passes through the two known points. The line thus established may be prolonged, by the methods already explained, from one of the given points to the other, and the line may then be chained.

To establish and mark the line between two points not mutually visible different methods may be used, depending on the distance between the points, the kind of obstacle that obstructs the view and the accuracy necessary to fulfill the purpose of the survey.

If the given points are several miles apart it will be best to make a survey by running an instrumental traverse from one point to the other along the most convenient route. By plotting this traverse the direction and length of the straight line joining the two points may be determined. The direction being known, the line may now be ranged out and marked on the ground if desired. Thus (Plate 14, Fig. 3), the direction and length of the line joining the two points A and B are required. The ground between them is undulating and partially wooded, so that the view is obstructed, and standing at either point, the exact direction of the other is not known. Run the traverse A, 1, 2, 3, 4, B, with transit, compass or plane table and plot the survey. (Plate 14, Fig. 4.) Draw the line a b. The scale of the plot gives the distance (2½ miles), and the protractor gives the bearing (N 50° E). If the line is to be staked out on ground, set up the instrument at A and point it in the known direction (N 50° E). Prolong this direction by clearing out the line and setting stakes at convenient intervals. If the determination of the direction was not accurate the prolonged line will not pass exactly through the point B, but when a point opposite B is reached the error can be determined and the line readjusted by moving the stakes to their correct positions on the line A B. Having cleared out the line and marked it at points that are mutually visible, the line may be chained and the distance determined with greater accuracy than was attained by traversing. For shorter distances the following method may be used:

Let A and B (Plate 15, Fig. 1) represent the two given points which by reason of intervening hills, or trees, or buildings, are not mutually visible. Starting at either point, as A, estimate as nearly as possible the direction toward the point B and range out a trial line, leaving stakes d, e, f, etc., at measured intervals. Arriving at point C where B is seen at right angles to the trial line, erect the perpendicular C B and measure it. Then correct the position of each stake by moving it on a perpendicular to the trial line over a distance that is proportional to its distance from A. Thus, k is moved to k′ making $k\,k' = \frac{A\,k}{A\,C}\,BC$, etc. This method may be used when an instrument is not available.

If the two points are on opposite sides of a hill or ridge, from the top of which both points can be seen, two rodmen may range each other in by successive approximations till both are on the desired line. Thus (Plate 15, Fig. 2), A and B being two given points separated by a ridge, the first rodman at C ranges in the second rodman at D by sighting at B. Then the second rodman ranges in the first at C′ and so on till both are on the line A B.

In any case, after the line has been marked at points that are mutually visible, the chaining may be done as already described.

With ordinary care in chaining an error of 1 in 1000 should not be exceeded. With greater care and by using a spring balance to regulate the pull and applying the correction for mean temperature, the error in chaining should be reduced to 1 in 5000 at least.

For ordinary purposes the chain is the most satisfactory and convenient of distance measuring instruments. It is durable, and may be dragged over rocky ground or through brush without injury; it is easy to use and rapid in operation. Its decimal divisions make the record simple and facilitate computations.

The only care that it requires is to wipe it dry after use and oil it to prevent rust. It should be tested from time to time and adjusted to standard length for normal pull and temperature.

For many purposes, and especially for use over broken ground, the half chain of 50 links is preferable to the full chain. In selecting a chain, get one of strong steel wire, not smaller than No. 15 gauge, and be sure that the rings are brazed into closed circles. If not brazed they will open and the chain will part. If the rings are oblong in form they will wear always at the same point and the chain will be lengthened with but little use. A bent link, when straightened, is permanently elongated. Frequent bending and straightening links will therefore lengthen the chain.

A novice in surveying is inclined to ignore the precise methods prescribed in text books and the precautions recommended therein as being trivial and unnecessary. A single attempt at closing a traverse, or measuring a long line whose length is accurately known, will convince him of his error. Haphazard methods improvised by the inexperienced, will result only in absurdities. Nothing but a carefully devised system, thoroughly carried out in all its details, will yield even fair results. For good results, infinite care, attention and forethought are necessary.

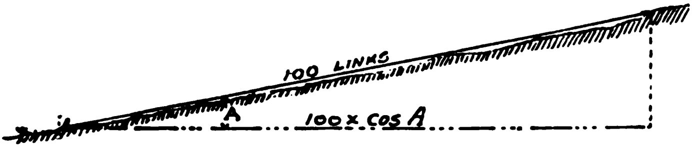


FIG. 1.

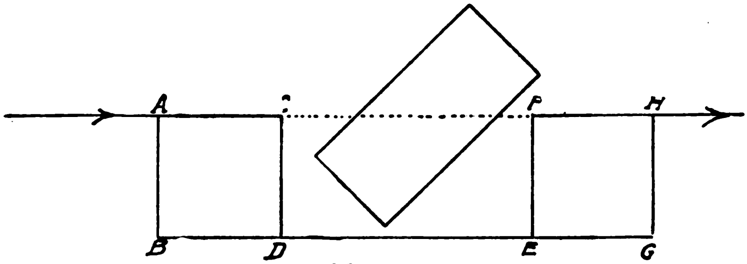


FIG 2

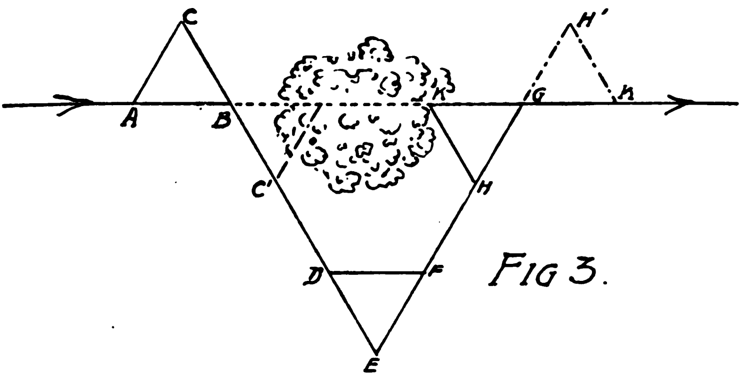


FIG 3.

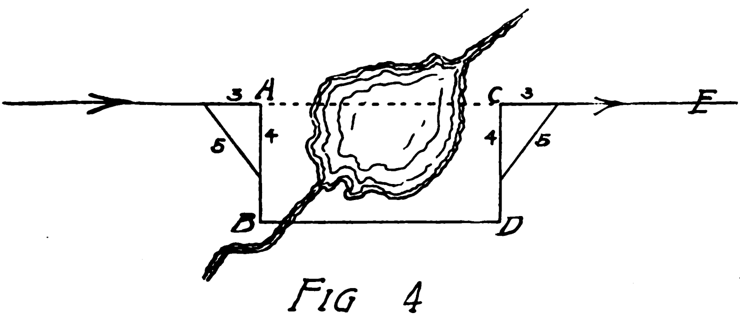


FIG 4

PLATE 13

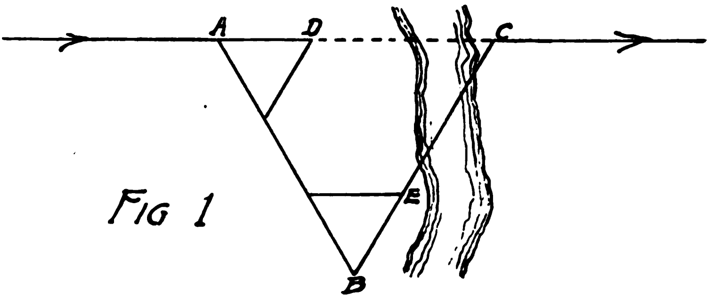


FIG 1

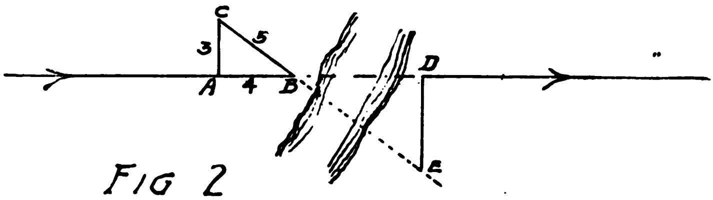


FIG 2

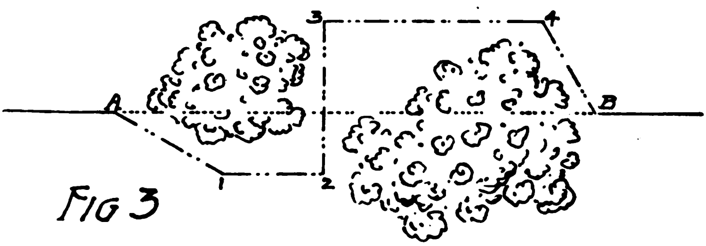


FIG 3

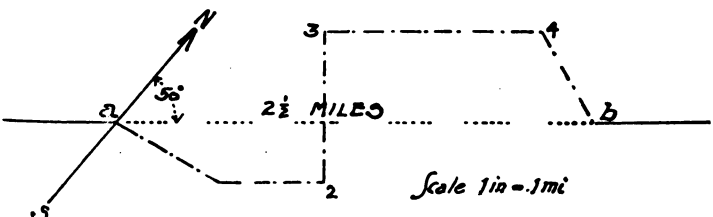


FIG 4

PLATE 14

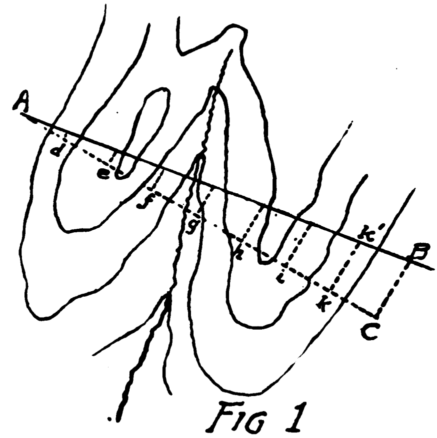


FIG 1

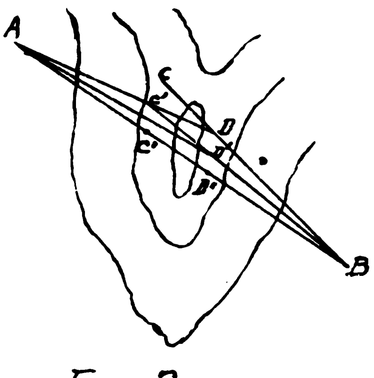


FIG 2

FIG 3

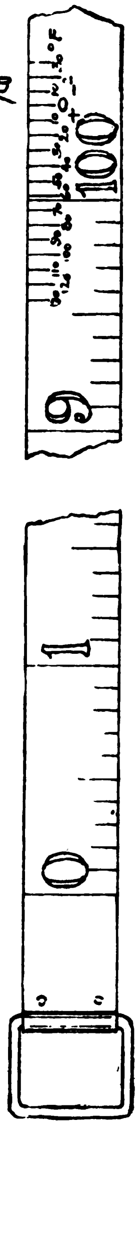


PLATE 15.

CHAPTER XIII.

THE STEEL TAPE.

The Steel Tape is a narrow ribbon of spring tempered steel. Some tapes are marked by etched lines and numbers, others by brass sleeves.

Tapes are made in any desired length up to 1000 feet, and are graduated in any divisions that may be ordered.

The Surveyor's chain-tape is 66 feet long and is divided into 100 links.

The Engineer's tape is usually 100 feet long and is graduated into feet, tenths and hundredths.

The Architect's tape is 50 or 100 feet long and is graduated into feet, inches and eighths of inches.

The longer tapes, 400 to 1,000 feet in length, are usually marked only at the 10 ft. or 100 ft. intervals, with smaller divisions into feet and tenths of a foot near the ends, and sometimes also near each 100 ft. mark.

Tapes graduated to meters, yards, varas, or any desired unit may be obtained.

For convenience in carrying and handling, tapes are wound upon a reel either in a leather case or in an open frame. The latter is to be preferred because it permits the evaporation of moisture and thus prevents rusting.

The steel tape is the most accurate and convenient of instruments for measuring distances. The makers will furnish a record of the test applied in standardizing the tape, giving the pull in pounds necessary to stretch the tape to standard length at a stated standard temperature (generally 62° Fah.), when supported throughout its whole length or when supported only at its ends, or when supported at stated intervals.

In making measurements in the field, the same conditions of pull and supports as were used in the test should be applied. The stretch or elongation due to pull may be made to balance the shortening due to sag, and there will remain only the temperature and slope corrections to be applied.

The *temperature correction.* Steel expands or contracts 0.000065 of its length for a change in temperature of 1° Fahrenheit. With a rise in temperature it is lengthened, and with a fall in temperature it is shortened. The standard temperature of tests is usually 62° Fah. At other temperatures the tape, when properly stretched and supported, will not have the length that is marked on it, and its true length must be determined. It may be accepted as an axiom that a 100 ft. tape is not 100 feet long.

Let L_s = tested length of tape at standard temperature. This is the length that is marked on the tape.

T_s = standard temperature = 62° Fah.

T_t = temperature of tape when used.

and L_t = length of tape at temperature T_t when properly stretched and supported.

Then

$$L_t = L_s + 0.0000065 \ (T_t - T_s) \ L_s$$

Or

$$L_t = L_s \, [1 + 0.0000065 \ (T_t - T_s)]$$

Rule:—Subtract algebraically the standard temperature from the temperature of the tape when used. Multiply the result by 0.0000065. Add unity algebraically, and multiply the result by the standard or marked length of the tape. The result will be its true length, that is, the length of a straight line joining the two points at which the end marks of the tape were held, provided that the tape is pulled and supported as it was in the original test.

When the tape is applied a number of times successively in measuring a long line it is not necessary to apply the reduction for temperature to each tape length, but only to apply the reduction for mean temperature to the sum of all the tape lengths. Thus, if six lengths of a 100 ft. tape have been applied in measuring a straight line, with temperatures of 68°, 68½°, 69½°, 70°, 71° and 71½° respectively, the mean temperature is 69¾°, and the length of the line is

$L = 600 (1 + 0.0000065 [69\tfrac{3}{4} - 62]) = 600.030$ ft.

When a partial tape is used to complete the measurement, and to close on the objective point it must be reduced separately, since each temperature must be given a weight equal to the length of tape to which it applies. Thus, if a length of tape marked 46.35 ft. were applied with a temperature of 73° in addition to the six full tapes, the corrected partial tape would be—

$$L = 46.35 (1 + 0.0000065 [73 - 62]) = 46.353 \text{ ft.}$$

giving a total distance of 646.383 ft.

Some tapes are marked by the maker with a temperature correction; that is, there are different 100 ft. marks corresponding to different temperatures. The 100 ft. mark moves 0.0065 of a foot for a change in temperature of 10° F. In using such a tape the temperature is read and then the mark which is numbered with the corresponding temperature is used as the 100 ft. mark (Plate 15, Fig. 3).

With such a tape it is not necessary to make the temperature reduction except for partial tape lengths.

The refinements of using a tested tape with a pull, measured by a spring balance, that will compensate for the sag, and of applying the correction for temperature, are adopted only when a distance is to be measured with extreme accuracy, as in measuring a base line for an important triangulation survey.

To measure a base line with an error less than 1 in 50,000.

Such accuracy is attainable only by exercising the greatest care and eliminating every possible source of error, especially such as are accumulative. The work should be done in calm cloudy weather or at night—never in bright sunlight. When the sun is shining the mean temperature of the tape cannot be determined.

The base line party should consist of one chief who also acts as recorder, one transitman, one rear rodman, one front rodman, one rear marker, one front marker, one axeman, one man to read thermometers and one teamster; total, nine men.

The most convenient tape to use is a steel tape, 100 ft. long, graduated to feet, and tenths and hundreds of feet, with the zero and 100 ft. marks on the tape several inches from the ends, and with a loop or handle at each end. The tape must have been carefully tested by the makers, or by the Bureau of Weights and Measures at Washington, or by the Mississippi River Commission, St. Louis; or by the U. S.

Lake Survey Office, Detroit; or at some college or university where standards of measures are available for comparison. The certificate of the test should give the pull in pounds necessary to stretch the tape to standard length (100 ft.) at standard temperature (62° Fah.) when supported at its ends, and similar tests should be reported for supports at the 0, 25, 50, 75 and 100 ft. marks respectively. These last are needed when partial tape lengths are used. The full tape should be used as much as possible.

The equipment consists of the tape described, a transit, posts 4"x4"x3' to 5', one for every 100 ft. of base line; two range poles, one level rod, one standard spring balance, two standard Fah. thermometers; one axe, one maul, one shovel, one scythe, zinc plates for tops of posts, one hammer, one hand saw, one try square; tacks, note books, pencils and one wagon.

If the party is in camp the usual camp equipage will be required.

PROGRAM OF OPERATIONS.

The base line stations, that is, the points at the ends of the base line, should be marked by permanent stone monuments, sunk below the frost line. The geometric point at each station should be marked by the intersection of two fine lines cut in the top of a copper bolt set in the stone. The horizontal distance between these two points is required.

It cannot be expected that the ground between the two base stations will be level and open. Generally it will be sloping or undulating, and obstructed by trees, weeds, brush, fences, etc. It is only on a straight line of sea beach, or on a long tangent of a railroad, or on a level barren plain, or on the ice of a Northern lake or sound, that the ideal conditions of a straight and unobstructed level base line can be attained. The first requisite for a base line is that when the line is cleared a signal pole erected over each station shall be visible from the other and from adjacent triangulation stations. The second requisite is that the ground between the stations shall be fairly smooth, open and unobstructed. These conditions must be considered when selecting the base line.

Select one station, A, as the origin, clear out the line, and erect a signal pole or target at the other station, B, centered and plumbed over the geometric point. Set up the transit at station A and point it at station B. The transit is then used to line in the posts at 100 ft. intervals along the base, and to establish the grade in which the tops of the posts are cut. On undulating ground the positions for the posts will soon be lost to view from station A, and the grade in which the tops of the posts should lie, will change. In either case the transit must be moved forward to a point on the line where a further view can be had to continue the line and grade.

At station A, orient the transit by sighting at station B, and clamp the plates. Line in the posts as far as they can be seen and drive them vertically and firmly, with their centers on line, and 100 feet apart, measuring with care, but not at this stage with the greatest accuracy.

Turn the telescope up or down to a position parallel to the slope of the ground, and clamp it. In Plate 16, Fig. 1, it is assumed that the first slope is 2°. Measure the height of telescope above the mark on the monument at station A, and set the target on the level rod at the corresponding reading. The rodman will then hold the rod against the face of each post in succession at a height indicated by the transitman, that will place the target in the line of sight. A line at the bottom of the rod is then marked on the post as the cutting line. When sawed off at these lines the tops of the posts will be on a line of uniform slope.

When the slope of the ground changes, the tops of the posts can no longer be maintained in the original slope and a new grade line must be established, as at station 5 (Plate 16, Fig. 1). The transit is moved to this station and the new grade is established as described for station A. The setting and sawing of posts to line and grade respectively is continued in the same manner over the different slopes, till station B is reached. The last grade line must pass through the mark on the monument at station B.

Each portion of the line that has a new slope or grade must be considered as a separate section, to be treated independently in the reduction to the horizontal. Each portion of the line that is measured with a partial tape length must be considered as a separate section, to be treated independently in the reduction to standard temperature. It is best, therefore, to make the "slope section" as long as possible, and to make the "partial tape sections" as few in number as possible in order that the reductions may be simplified. If the same grade and the full tape could be used throughout the line, there would be but one section, and only one reduction to the horizontal and to standard temperature would be necessary. The grade line should not be more than three and one-half feet, nor less than one foot above the ground at any post. When these limits are exceeded a change of grade becomes necessary. While the transit is still set for a grade the posts of that grade are capped by a plate of zinc tacked on, and a fine mark is scratched on the zinc, exactly "on line," as determined by the telescope.

Measuring the Line.—When the posts of the first section of the line have been set on line and cut to grade at trial intervals of 100 feet the measurement may begin.

The rear rodman fastens the zero end of the tape to his rod or staff by means of a loop of strong cord and uses the staff as a lever to steady the tape and oppose the pull exerted by the front rodman. By raising or lowering the loop on the staff, and by properly inclining the staff, the edge of the tape at the zero mark may be made to coincide exactly with the cross mark on the monument or station post.

The rear rodman devotes his whole attention to holding the staff steady and bringing the zero mark on the tape exactly to the mark on the post.

The rear marker watches the mark on the tape and assists the rear rodman to bring it exactly to the station mark. When he sees that the coincidence is exact he calls out "Mark" to the front marker. The rear marker and rodman must take care that the post is not touched except by the tape, and that the tape barely touches the zinc when drawn across its top. The post must not be disturbed in the slightest degree. Failure to observe this precaution has caused many errors in base-line measurements.

The front rodman attaches the ring of the spring balance to his staff by a loop of strong cord and then hooks the balance into the loop of the tape. He uses the staff as a lever to stretch the tape. By raising or lowering the loop and properly inclining the staff, he causes the edge of the tape to coincide with the scratch on the zinc, and at the same time pulls out the spring balance to the reading required by the certified test of the tape. His whole attention is devoted to keeping the tape on line and maintaining the required pull on the balance. When the rear marker calls out "Mark," he repeats the word *if at that instant the balance indicates the proper pull.* If not he says "No," and calls for a new trial.

The front marker fixes his attention on the 100 foot mark of the tape and holds a sharp steel point, as a knife point or sharp scratch-awl at that mark. When the signal, "Mark" of the rear marker is repeated by the front rodman, he sets the steel point in the zinc exactly at the 100 foot mark of the tape. The tape is moved a little to one side, and he places a straight edge crosswise against the point and scratches a fine cross line on the zinc, *taking care not to disturb the post.*

The tape is then stretched again in the same manner, and at the signal "Mark" the position of the cross line is verified.

Although a post may appear to be solidly and firmly driven in the ground, still a slight knock or pressure will displace it and vitiate the measurement. Hence, the precaution prescribed against touching the post after it has been marked.

When a post has been marked and verified, the tape is carried forward and a new post is marked in the same manner.

The axeman examines the tape throughout its length as soon as it is stretched, and sees that it is without twist and that it hangs clear of all obstructions. If it is fouled by weeds, grass, or brush, he clears the obstruction away. If it touches the ground, the earth must be dug away to clear it. When the tape is carried forward, he carries it at the middle point to keep it from dragging on the ground.

The thermometer man provides himself with two light rods on which to support his two thermometers. When the tape is stretched he plants one rod at the 25 ft. point, and the other at the 75 ft. point and hangs the thermometers on the rods at the height of the tape, but not touching it. While the marking is being made, he takes and records two or three readings of each thermometer, finds the mean reading and reports the result to the chief of party.

The chief of party supervises the work and sees that all the details are carefully attended to. He should watch the rear mark during the first reading, and the forward mark and the spring balance during the verification of each tape length; he should see that all obstructions have been cleared away so that the tape hangs clear, and he should read the thermometers to check the report of the thermometer man. It is only by constant watchfulness and verification that he can secure the results for which he is responsible. If he is careless and inattentive, it is certain that the members of his party will be more so.

The record is best kept in diagrammatic form, each separate "slope section" or "partial tape section" of the line being represented by a sketch at the top of a separate page of the note book.

Each slope section is represented by a profile line (Plate 16, Fig. 2) reading from left to right, and marked with the number of the section in Roman characters, the included stations in Arabic characters, the slope in degrees (+) for rising and (—) for falling, and with the mean temperature Fah., for each tape length, and mean temperature of section.

$$\text{Mean Temp.} = 40\tfrac{1}{4}^\circ \text{ F.}$$

$$\cos 3^\circ = 0.99863$$

$$40.25^\circ - 62^\circ = -21.75^\circ \text{ F.}$$

Reduction for slope and temperature:

$$L_t = 400 \times 0.99863\, [1 - (0.0000065 \times 21.75)].$$

$$\begin{array}{r} -(0.0000065 \times 21.75 = -0.000141375 \\ +\,1. \\ \hline 0.999858625 \end{array}$$

$$400 \times 0.99863 \times 0.99986 = 399.3961 = L_t$$

The method of reduction is indicated in the foregoing example. To reduce to the horizontal multiply the measured length by the cosine of the slope, and to reduce to standard temperature multiply also by

$$1 + [0.0000065\, (T_t - T_s)]$$

The result is the horizontal length of the section, and if five decimal places be retained throughout the computation, the error introduced in computation will not exceed 1 in 100,000.

For a partial tape section the record and reduction are made in a similar manner (Plate 16, Fig. 3); thus:

$$\cos 4^\circ = 0.99756$$

$$76^\circ - 62^\circ = 14^\circ \text{ F.}$$

$$L_t = 58 \times 0.99756 \times [1 + (0.0000065 \times 14)]$$

$$\begin{array}{r} 0.0000065 \times 14 = 0.000091 \\ + 1. \\ \hline 1.000091 \end{array}$$

$$58 \times 0.99756 \times 1.000091 = 57.8637 = L_t$$

When all the sections of the base line have been thus measured, recorded and reduced, the reduced lengths of sections are added together to get the horizontal length of base line.

When partial tape lengths must be used, it is best to select a length of 25, 50, or 75 feet, if the tape has been tested at these points.

In closing on station B, use first a tested length of 0, 25, 50 or 75 feet, whichever comes nearest to the station mark, and then measure carefully the small remaining distance, with a pull estimated by comparison with the pull required for 25 feet. This small section is to be recorded and reduced separately like the other partial tape sections.

There are other methods of measuring base lines with the tape, but the method above described takes account of the principal sources of error, and if carefully followed will give the proposed result, viz., the length of base line with an error less than 1 in 50,000.

Measuring Lines With the Tape.—It is only in measuring the base-line of an important survey that such precise methods as have been described are necessary. For ordinary measurements, a method similar to that described for the chain will suffice

The tape is stretched by hand with an estimated pull; tape lengths are marked on the ground by marking-pins or small stakes; on sloping ground the tape is held horizontally as well as the eye can determine, and the end points are referred to points on the ground by means of a plumb line. Careful measurements thus made on a main traverse should not err by more than 1 in 5000. Otherwise, the measurements may be made on the slope; and if the slope exceeds $1^\circ 00'$, the usual reduction to the horizontal should be made by determining the slope and multiplying by its cosine.

For hasty measurements made to "fill in" a survey by side shots, an accuracy of 1 in 1000 will suffice.

Measuring Distances With the Stadia.—This is the most rapid, convenient and accurate method available for running traverses and "filling in" when the transit or plane table is used. The principle of the stadia and its use are explained in connection with the transit. The error of a single measurement may be as great as 1 in 600, but in many measurements the compensation of errors will probably reduce the total error to about 1 in 3000.

Pacing.—The paces of a man who walks steadily at a marching gait are quite uniform, and if the length of a pace be known it forms a convenient unit of measure for rough and hasty surveys or sketches.

It is not the length of any single pace that is required, but rather, the average horizontal length of a pace when walking over varied ground, up hill and down hill, on roads and on grass. To determine this average length of pace, measure a course of 1000 to 2000 feet over varied open ground that is fairly representative of the country to be covered in subsequent work. Mark the ends of the course and pace the distance several times in both directions, counting the number of steps each time. The length of the course in inches divided by the number of paces will give the length of the pace in inches. If the results of the several trials differ by more than two per cent., repeated trials should be made with greater attention to uniformity of step and cadence, until by practice the desired regularity is attained. The gait should be a natural rapid walk that can be maintained for a day's march.

A following wind lengthens the pace and a head wind shortens it. Mud and loose sand shorten the pace. On slopes less than four degrees the pace is not materially affected. On steeper slopes the pace is shortened whether moving up or down, but if the pace has been determined on a course of varying slopes such as are met with in the field, no further correction for slope will be necessary. On steep slopes pacing cannot be relied upon and some other method of determining distances should be used, such as triangulation, or the range finder.

The paces of different men vary from about 27 inches to 37 inches as the extremes, but for most men the length of pace is from 30 to 33 inches.

Many prefer to count *strides* instead of paces. A stride is the distance between two successive positions of the same foot, and its length is therefore twice the length of a pace.

In counting paces or strides mistakes sometimes occur through leaving out or adding entire tens or hundreds. To avoid such errors it is necessary to fix the attention on the count, and this prevents observation and study of the ground passed over. It is well, therefore, to use some form of mechanical counter for recording the count.

The passometer is a small tally register that is held in the hand. A lever is pressed by the thumb or finger at every step or stride, and the number is indicated on a dial. The motion of the hand soon becomes a sub-conscious operation that requires no attention.

The pedometer registers the vibrations of a carefully poised weight that is balanced by a spring. The shock of planting the foot at each step produces a vibration of the weight, which is recorded on the dial. The spring should be adjusted to suit the cadence, by tightening for a rapid pace and loosening for a slow pace. The instrument has the shape of a watch and may be carried in the pocket. It is not as reliable as the passometer.

Pacing is the method generally used for measuring distances in making a rough survey or "sketch," such as a road or position sketch or reconnaissance on foot. In this work the hand compass is generally used for determining directions, with an error that should not be greater than one degree for a single observation. The tangent of one degree is, roughly, $\frac{1}{60}$. The error in pacing should not exceed 1 in 60. Therefore, the determination of distances by pacing is equally as accurate as the determination of directions by the hand compass. When the hand compass is used for directions, the pace is just as good as a steel tape for distances.

This same principle applies in all methods of surveying, viz: When a certain limit of error obtains in the determination of directions, the same limit of error is permissible in the measurement of distances, and vice versa. Thus the transit and the steel tape are used together, the surveyor's compass and the chain are used together, and the hand compass and the pace are used together.

The step or stride of a horse at a steady walk or trot is more uniform than that of a man. It is also less affected by wind, mud, sand or slope, and is a good unit of measure, when its average horizontal length has been determined. This length is determined by riding over a measured course on varied ground at the regular gaits and counting the steps or strides. A number of trials are necessary in order to gait the horse to a steady walk and trot that can be maintained in subsequent work. When the average length of the step or stride at the walk and trot has been determined any distance may be measured by riding over it at the adopted gait and counting the steps. A passometer should be used to keep the count in order that the attention may be devoted to careful observation of the country traversed.

When a wheeled vehicle is used as a means of transportation, the revolutions of a wheel afford a means of measuring distances. The unit of measure is the distance passed over by the wagon during one revolution of the wheel. To determine its length the wagon is driven over a measured course on a road similar to those that will be traversed in subsequent work, and the revolutions of the wheel are counted. The length of the course divided by the number of revolutions gives the distance covered in one revolution. When this distance has been determined, any distance traversed by the wagon may be measured by counting the revolutions of the wheel. To obtain the horizontal distance, the measured distance on each slope must be multiplied by the cosine of the slope.

A direct measurement of the circumference of the wheel is not a good determination of the unit of measure in this case, because the wheels slip more or less, and on rocky, sandy, or muddy roads the slip is considerable. The front wheel should be used because the hind wheels are sometimes locked by the brakes in going down hill.

A rag of white cloth tied to a spoke is a convenient mark to indicate successive revolutions.

The Odometer is an instrument that automatically registers the revolutions of a wheel to which it is attached. In one form the mechanism is contained in a leather case and is strapped to the spokes of the wheel. A weighted frame hangs freely on a central axis in the case and remains vertical while the wheel and case turn. Worm gearing communicates a slow motion to the numbered dials in the frame and thus registers the number of revolutions. To read the dial the case must be opened and the frame withdrawn.

In another and better form the recording apparatus is attached to the front axle and is operated by a lever which is raised at each revolution by a cam or eccentric on the hub of the wheel. The cover of the dial may be left open and readings taken at any time without dismounting.

MEASURING DISTANCE BY TIME.

If a man moves on foot or horse back, or by wagon or boat, at a uniform rate, the time during which he so moves may be taken as a measure of the distance traversed, provided that the distance covered in a unit of time is known. On land this method of measuring distance is used principally in mounted reconnaissance, and for this purpose the rate of the horse must be determined. Ride several times at a walk and at a trot over a measured course in both directions and note the time for each trip. Find the average time for each gait. The length of the course divided by the number of minutes taken to traverse it gives the distance corresponding to one minute, and the minute becomes a known unit of measure when the same horse is used at the rated gaits in subsequent work. The horse should be rated by the rider who is to use him in the reconnaissance, since a change of riders changes the rate of the horse. After careful gaiting and rating, the error in measuring distances by this method should not exceed 1 in 50, or 2 per cent., an ordinary roads.

For convenience and accuracy in timing distances a stop watch is essential. The pattern that is alternately started and stopped by one lever and is set back to zero by another is best suited for this work, since it may be stopped and read at every halt and started when the advance is resumed. At the station halts it is set back to zero, and it therefore always indicates the distance in minutes from the last station.

To attain equal accuracy in the determination of directions the hand compass or the sketching case is used, and for a like approximation to differences of elevation the clinometer or the slope board will suffice.

At sea a similar method, called "dead reckoning," is applied for tracing the course of a vessel. The speed is determined by the "log" in miles per hour, and time thus becomes the measure of distance. Directions are given by the compass. There are no differences of elevation to be considered until the harbor entrance is reached, and then the stage of the tide is the vertical coördinate that shows whether the vessel is high enough to clear the bar. Check points corresponding to triangulation stations are furnished by the daily sextant and chronometer observations for latitude and longitude. These methods have become familiar to many of our officers in their voyages to the Philippines. The principles involved are identical with those applied in surveying. Dead reckoning is traversing, and the determination of latitude and longitude is spherical triangulation. It is the sextant and chronometer, more than the compass and log, that brings a vessel surely to her haven.

PLATE 16

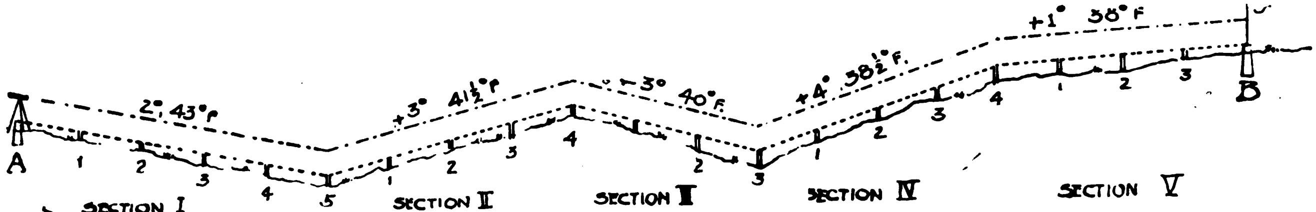


FIG 1

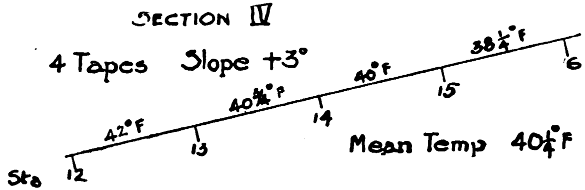


FIG 2

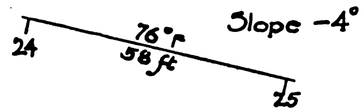


FIG 3

CHAPTER XIV.

TRIANGULATION.

Triangulation is an application of the triangular system of coördinates in the accurate determination of the relative positions of selected points, well distributed over the area considered. Points so determined are used as control points to check and correct the results of the less accurate but more rapid and economical methods ordinarily used in surveying.

Let the point A, Plate D, be selected as the initial point of the survey. A second point, B, is first determined by measuring its azimuth, horizontal distance, and difference of elevation from A. The line A B is the *base line*, and the horizontal distance from A to B is the length of the base line. The operation of measuring a base line is described in Chapter XIII.

A third point, as C, may now be determined by measuring two other parts of the triangle A B C, either the two sides a and b, or two of the angles as A and B. Since it is easier to measure angles than to measure distances, angles are usually measured.

Measure the angles A and B with a transit, and solve the triangle as follows:

$$C = 180° - (A + B)$$

$$a = \frac{c}{\sin C} \sin A \ldots\ldots\ldots\ldots (1)$$

$$b = \frac{c}{\sin C} \sin B \text{............} (2)$$

in which A and B are the measured angles and c is the measured length of base line, C is the required angle, and a and b are the required lengths of the sides opposite the angles A and B respectively. All of the parts of the triangle being thus found, the point C is fully determined in plan and may be plotted.

To check the accuracy of the angle measurements, the third angle at C should also be measured with the transit. The sum of the angles of a triangle is 180°. If the sum of the three measured angles is not 180°, the discrepancy is due to errors of measurement, and suitable corrections are applied to give angles whose sum shall be 180°.

If the angles have been measured with equal care, it is assumed that equal corrections should be applied to all, and each angle is changed by one-third of the total error so that their sum shall be 180°.

For example, suppose that the angle measurements give the following results:

$$\begin{array}{rcl} A = & 41^\circ\ 26'\ 30'' + 10'' = & 41^\circ\ 26'\ 40'' \\ B = & 93^\circ\ 58'\ 15'' + 10'' = & 93^\circ\ 58'\ 25'' \\ C = & 44^\circ\ 34'\ 45'' + 10'' = & 44^\circ\ 34'\ 55'' \\ \hline \text{Sum} = & 179^\circ\ 59'\ 30'' + 30'' = & 180^\circ\ 00'\ 00'' \end{array}$$

The sum of the measured angles is too small by 30″, therefore 10″ must be added to each of the observed angles to give the corrected values which appear in the last column, and whose sum is 180°. It cannot be claimed that these corrected values are correct, but they comply with a known condition, which is better than any observation.

If the angles have not been measured with equal care, weights are assigned to the observed angles expressing the estimated accuracy of the several measurements, and the total error is divided into parts that are inversely proportional to the weights.

Thus, in the foregoing example, suppose the weights in order are respectively 3, 2 and 6. Then the total error, 30″, must be divided into parts proportional to $\frac{1}{3}$, $\frac{1}{2}$ and $\frac{1}{6}$, or as 2 : 3 : 1, giving + 10″, + 15″ and + 5″ as the corrections, and

$$\begin{aligned} A &= 41^\circ\ 26'\ 40'' \\ B &= 93^\circ\ 58'\ 30'' \\ C &= \underline{44^\circ\ 34'\ 50''} \\ \text{Sum} &= 180^\circ\ 00'\ 00'' \end{aligned}$$

as the corrected angles.

The corrected angles are used in solving the triangle to find the sides a and b.

In like manner some other point as D, may be determined by measuring the three angles, correcting them to distribute the error, and solving the triangle A B D.

Any two of the determined points may be used as the ends of a new base for determining a new point. Thus, from A and C the point F may be determined by the triangle A C F. From A and D the point G may be determined, and the triangle F A G may be completed by measuring its three angles. Whenever, as in this case, a point, such as A, is surrounded by measured triangles, a further check is introduced by the condition that the sum of all the angles around this point must be 360°, and all the angles of the five triangles may be readjusted so that this condition shall be satisfied without disturbing the first condition.

If the points C and D be used to determine a point, E, the side C D is first found by solving the triangle C A D, of which two sides and the included angle are known. If A′ represent the included angle,

a′ the required side,

c′ and d′ the known sides,

then

$$a' = \sqrt{c'^2 + d'^2 - 2\,c'd' \cos A'} \qquad \text{(3)}$$

Having thus found the side C D, the triangle C D E may be solved by (1) and (2) to determine the point E.

The system of triangles may be extended to cover any desired area.

The method of adjustment above indicated is the simplest and most obvious, and involves only the conditions that the sum of the angles of a triangle is 180°, and that the sum of the angles about any point is 360°. As each triangle, or group of triangles, is adjusted, the result is considered final and each new point is assumed to be accurately determined. This assumption is approximately correct for a small plane survey, say 100 square miles or less in extent, made with care and with a good transit. For important and extensive surveys, covering thousands of square miles, geodetic conditions are involved, taking cognizance of the ellipsoidal figure of the earth, and requiring primary triangulation with precise instruments and rigorous adjustment by the method of least squares.

Primary geodetic triangulation is an undertaking of such magnitude that only governments can indulge in it. No private or local interest would justify the expense. But in regions that have been covered by primary triangulation, all important local surveys should, when practicable, be based upon and controlled by the government surveys.

In primary triangulation the stations may be from 10 to 100 miles apart and do not serve the purpose of control and adjustment for a small survey. The stations of the primary system are, therefore, connected by chains of smaller triangles corrected and adjusted to close on the primary stations. This constitutes "secondary triangulation."

For the survey of a limited area or for "filling in" an extended survey, the stations of the secondary system may be connected by "tertiary triangulation," giving stations only one to three miles apart. Traverses may then be run with transit, or plane table, or surveyor's compass, and by beginning and closing every traverse on triangulation stations each short traverse of from one to three miles may be adjusted and corrected. Errors and mistakes in traversing will not be propagated throughout the survey, but will be detected and eliminated at every closing station.

When primary and secondary systems of triangulation are not available, as is usually the case, the control of a plane survey of a limited area is secured by a system of small triangles corresponding to tertiary triangulation and adjusted by the simple method that has been described.

In measuring the angles each observation is repeated a number of times and a mean of the results is taken as the observed angle. One methed of repeating the measurements is as follows:

To measure at station A the angle C A B.

Record:

At station A, 5 observations.

	Vernier A.		Vernier B.
On Sta. C	$23^\circ\ 14'\ 20''$		$203^\circ\ 14'\ 20''$
On Sta. B	$230^\circ\ 26'\ 40''$		$410^\circ\ 27'\ 00''$
$10 \times$ angle $=$	$207^\circ\ 12'\ 20''$	$+$	$207^\circ\ 12'\ 40''$
	$= 414^\circ\ 25'\ 00''$		
Angle C A B $=$	$41^\circ\ 26'\ 30''$		

Set up the transit at station A, clamp the lower plate, point at the left hand station, C, clamp the upper plate, read both verniers and enter the read-imgs in the record as shown above. Unclamp the upper plate, point at B and clamp. This ends the first observation. Unclamp the lower plate, point at C and clamp. Unclamp the upper plate, point at B and clamp. This ends the second observation. Repeat these observations to include the fifth pointing at station B. Read both verniers. If a vernier has passed the 360° point once, add 360° to its final reading; if twice, add 720° to its reading. Enter the readings in the record under the first readings and find the difference of the readings for each vernier. The sum of these differences divided by twice the number of observations gives the required angle.

For greater accuracy a second set of observations may be made as follows: Shift the verniers so that a different part of the circle will be used, and make the initial pointing on the right hand station, B. Then turn with the upper clamp to the left and with the lower clamp to the right, to include the fifth pointing at station C. The remainder of the operation is the same as for the first set. The mean of the two results is taken as the required angle.

In practice, all of the triangulation stations are established and marked, including those of the base line. The stations must be selected with a view to covering the ground with well conditioned triangles and preserving mutual visibility between adjacent stations. A good station mark for temporary purposes is a piece of 2-inch pipe sunk about two feet in the ground and projecting one foot. Rods bearing different combinations of flags are set in the pipes as targets and distinguishing marks. When a station is occupied the rod is removed and a plug having a center mark in the top is inserted. Always replace the flag rod before leaving a station.

Measure the base line twice in opposite directions and take the mean of the two results. Determine the true azimuth of the base line by an observation on Polaris. Determine the difference of elevation of the base stations by leveling.

Measure all the angles of the selected triangles with five or ten repetitions of each angle, adjust the angles and compute the lengths of all the sides. Compute also the true azimuths of all the sides, the latitude and departure of each side, and the latitude distance and meridian distance of each station from an assumed origin.

To plot the triangulation stations, rule a sheet of drawing paper with fine pencil lines into accurate squares, such that the side of each square shall represent 100 feet, or 500 feet, or 1000 feet, etc., depending on the scale of the map. Assume a convenient origin and from it lay off the rectangular coördinates given by the latitude distances and meridian distances that have been computed. Mark the points thus plotted with the names of the corresponding stations.

When traverses have been run for filling in, and have been adjusted, by the method of latitudes and departures, to close on the triangulation stations, the same system of squares may be used in plotting the traverses.

VERTICAL ANGULATION.

For vertical control in a triangulation survey, the elevations of all the triangulation stations or of bench marks near them should be determined. This may be done by running lines of levels between stations, but it is easier and quicker, though not so accurate, to measure the vertical angles between stations and compute the differences of elevation. This latter method is called vertical angulation, or trigonometrical leveling, and is applied as follows:

Let A and B be any two stations, and

Let H = height of instrument at station A, that is, the height of the horizontal axis above the top of the stake or iron pipe.

A = the vertical angle or reading of the vertical circle when pointing at the *top* of the flag rod at station B.

R = height of flag rod at station B from top of rod to top of iron pipe.

b = horizontal distance from A to B.

d = difference of elevation or vertical distance from top of iron pipe at A to top of iron pipe at B.

then

$$d = H + b \tan A - R.$$

When the distance b is greater than one-half mile, the correction for curvature and refraction should be applied. If K represent the distance in miles, then $0.57\ K^2$ is the correction in feet, and is always positive. The above formula becomes

$$d = H + b \tan A - R + .57\ K^2.$$

A positive value of d indicates that station B is above station A, and a negative value of d indicates that station B is below station A. When the angle A is an angle of depression it is negative and the second term in the second member of the above formula becomes negative.

The difference of elevation from B to A is also determined by reading at B the vertical angle to the top of the flag rod A. The two results should be numerically equal with contrary signs. Their numerical mean is taken as the observed difference of elevation between the two stations.

In practice the vertical angles are read when the stations are occupied for reading the horizontal angles. Several readings should be made of each angle with telescope alternately direct and reversed. The record and reductions may be shown as follows:

At Station A, Vert. Ang. to B.

D.— 2° 24′		
R.— 2° 30′	$b = 7495$	$K = \frac{7495}{5280} = 1.42$
D.— 2° 22′		
R.— 2° 28′	$b \tan A = -318.54$	$.57\ K^2 = 1.15$
	$-R = -15.72$	$H = 3.78$
$A = -2° 26′$		
	A to B $= -334.26$	$+4.93 = -329.33$
$\tan A = -.0425$		

Similar record and reduction of observations at station B on A will give the difference of elevation from

B to A, and the mean of the two results is the observed difference of elevation.

Similar results are obtained for each side of all triangles, and may be adjusted by a method analogous to that used in adjusting the triangles.

For, the algebraic sum of the differences of elevation, taken in order around the sides of a triangle or other polygon, is zero. Therefore, the observed differences of elevation may be adjusted by corrections which shall make their algebraic sum, taken in order around the sides of each triangle, equal to zero.

Thus, in triangle A B C, Plate D, let the observed differences of elevation be

A to B,	— 329.33	— 0.49	=	—	329.82
B to C,	— 48.60	— 0.49	=	—	49.09
C to A,	+ 379.41	— 0.49	=	+	378.92
A to A,	+ 1.48	— 1.48	=		0.00

The total error is + 1.48, therefore the total correction is — 1.48, one-third of which, or — 0.49, is applied to each side, giving in the last column the corrected differences of elevation, whose algebraic sum is zero.

If the elevation of station A be known or assumed, as, say, 823.50, then B, 329.82 feet lower, has an elevation of 493.68, and C, which is 378.92 lower than A, has an elevation of 444.58.

In adjusting an adjacent triangle, as A C F, the corrected value, — 378.92, instead of its observed value, is used for the side A C and is not further corrected, one-half of the resulting total error being applied as a correction to each of the other two sides, C F and F A. When two sides of any triangle have been adjusted as parts of adjacent triangles no further adjustment is possible, as the difference of elevation on the third side becomes known.

In any system of triangles or polygons, that polygon in which the total error is greatest is first adjusted and the others follow in the order of the magnitudes of their total errors.

The corrected differences of elevation are used to determine the elevations of stations, beginning at one whose elevation is known or assumed, and the elevations thus determined are used to check and adjust the elevations in traverses that close on the triangulation stations.

TRIANGULATION SURVEY.

A complete topographical survey controlled by triangulation involves the following operations:

1. Select and mark suitable triangulation stations well distributed over the area to be surveyed, and forming fair shaped triangles with no angle less than 30 degrees. In some cases lines must be cleared of trees and other obstructions or high framed stations must be erected to give a view of adjacent stations. Two of the stations will be those of the selected base line.

2. Measure the base line twice in opposite directions and take the mean of the reduced results.

3. Determine the true azimuth of the base line or of any side of a triangle by an observation on Polaris.

4. Measure the horizontal angles in all the triangles, repeating five or ten times for each angle, and

measure vertical angles at each station to all adjacent stations.

5. Adjust the triangles, compute azimuths, lengths of sides, latitudes and departures of sides, and meridian and latitude distances of stations. With the rectangular coördinates thus determined plot the stations on coördinate paper (ruled in squares to scale).

6. Adjust differences of elevation on sides of triangles and determine elevations of stations. Mark these elevations at the plotted station points.

7. Run a system of traverses, each traverse beginning and closing at triangulation stations, and cover the area considered with a network of traverses such that all desired critical points shall be included.

8. Reduce the traverses to latitudes and departures and differences of elevation and adjust them to close on triangulation stations, finding the meridian and latitude distances and the elevations referred to the same origin as that used in the triangulation system.

9. Plot the traverses by rectangular coördinates and mark the plotted traverse stations with their elevations.

10. At each traverse station plot the sideshots with protractor and scale and sketch in the determined features. Mark the elevations of all determined critical points and interpolate and draw the contours.

11. Ink the drawing and finish the map by adding scale, magnetic and true meridians, title, date, legend, and border.

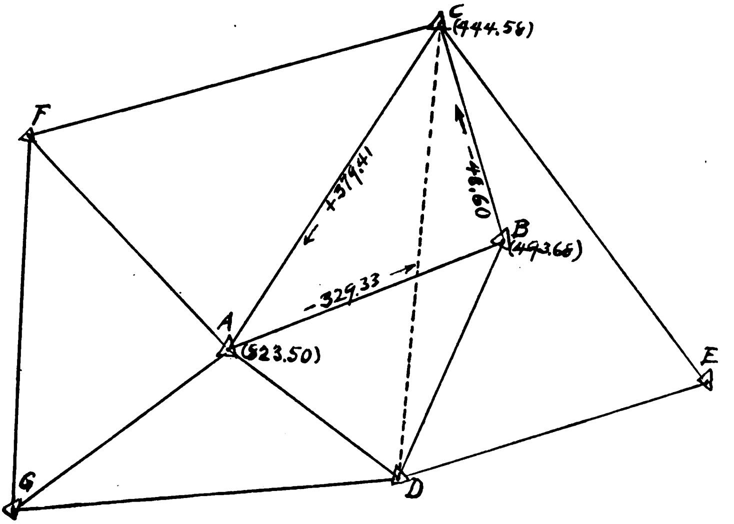


PLATE D.

CHAPTER XV.

SKETCHING.

The term sketching is applied to hasty methods of surveying that are used when considerable areas or great distances must be covered in a limited time. Necessity for the employment of such methods arises with imperative force in connection with military operations in a region which has not been mapped with sufficient detail for military purposes, or of which suitable maps are not obtainable. Like methods are employed in explorations in new countries and in preliminary reconnaissances made to select lines for new roads or railroads.

In sketching, the same principles are applied as in topographical surveying, but the simplest of means are used to assist the eye and mind in estimating directions, distances and elevations, and the hand in representing on paper the features thus determined. A knowledge of the principles of surveying is essential to a thorough understanding of the methods of sketching, and practice in surveying is the best preliminary school of instruction for the sketcher.

The surveyor should, by careful and conscious effort, constantly compare the appearance of distances, angles, slopes and heights as they are presented to him on the ground, with their determined values, and with their appearance as represented or represented on the map, keeping always in mind the scale of the map and the relative sizes and forms of corresponding features on ground and map. He will thus quickly acquire facility in estimating distances, angles and slopes with considerable accuracy, and will gradually attain a power of representing on paper, with almost unconscious effort, the forms and features which he sees on the ground. By repeated verification or adjustment of his estimates by the check of accurately determined values, he acquires correct standards of comparison that are attainable in no other way. An experienced topographer will need only a few determined critical points for control in sketching in all the intervening and surrounding features, forms and incidents of surface that lie within his unobstructed view. This implies that the map is drawn in the field and that the ground is studied while its shape and features are transferred to the map by the contours and other lines that represent them.

Plane table surveying is therefore the best practice for the sketcher. The surveyor should make a mental estimate of each distance, angle, slope, and difference of elevation before it is determined, and should, with light pencil touches, sketch on the map his estimate of the positions of points and of the shape of contours before they are actually plotted. He will soon find that his estimates have become quite accurate, and that he can dispense more and more with determined locations excepting for a few important control points around which he may fill in, by estimation and free-hand sketching, the representation of the intervening features.

Surveying also teaches the amount of detail that can be shown on maps of different scales. On a large scale map of say 12 inches = 1 mile (1 in = 440 ft.) with contours at 5 ft. V. I., all the minor inequalities of the ground can be shown by the contours, and buildings can be drawn to scale. Therefore when a map of this or larger scale is required, all of these minor details must be determined by measurement or by estimation. On a small scale map of say 3 inches = 1 mile, with 20 ft. contours, the smaller features of the surface cannot be shown, and buildings will be reduced on the map to mere dots. The surveyor or sketcher must learn to waste no time in determining small details that cannot be shown on the map and to confine his attention to the larger masses and general forms that will be apparent at the adopted scale.

To cover large areas or great distances rapidly this method of mapping by estimation, and by free-hand drawing must be applied to its fullest extent, with only such assistance in maintaining general control as is afforded by the use of hand instruments.

For military purposes, as stated in Chapter II, sketches may be classed, in general, as position sketches and road sketches. To these may be added a third class called route sketches made when hundreds of miles are to be covered in exploratory expeditions. These differ only in the amount of detail required and in the resulting scales:

Position sketch,	6 in. = 1 mi.	V. I. = 10 ft.
Road sketch,	3 in. = 1 mi.	V. I. = 20 ft.
Route sketch,	1 in. = 1 mi.	V. I. = 60 ft.

Sketches are made on foot, on horseback, and in wheeled vehicles.

Position sketches cover relatively small areas, or are divided up into small areas for individual sketchers, and require considerable detail. They are therefore usually made on foot.

Road sketches are made at the ordinary rates of marching and should cover from 15 to 20 miles of road in a day. They are therefore usually made on horseback.

Route sketches may cover many successive days' marches, and are made either on horseback or in wheeled vehicles.

The following instruments are used:

For determining angles and directions, the hand compass, either box or prismatic, the pocket sextant, the oriented plane table, or sketching case.

Distances are determined by pacing, or by timing a rated horse, or by counting the revolutions of a wheel. A "pace tally" operated by hand is convenient for recording paces either of man or horse; a stop watch is essential in measuring distances by time; and the odometer is used to record the revolutions of a wheel.

Slopes are measured with the clinometer or the slope board. Slope and distance give difference of elevation. Elevations are also determined by means of the aneroid barometer.

In plotting the work the paper is fastened on a small drawing board or on the sketching case. Directions are plotted by means of a small rectangular protractor, or, when the plane table method is used, by oriented board and pointed ruler. Distances are plotted by means of a working scale marked on the edge of the ruler or protractor. Contour points are spaced by means of scales of map distances marked on the edges of a card or ruler.

The above named instruments have been described in the chapters on surveying, with the exception of the sketching case, the aneroid barometer and the scales of map distances.

The *Sketching Case* is a small drawing board or hand plane table with attached compass and ruler. Rollers on opposite edges of the board stretch the paper and permit it to be shifted as the sketch progresses. The compass is used to orient the board. The ruler is attached to the board by means of a pivoted arm or frame so arranged that the ruler may be shifted to any part of the paper, pointed in any direction and clamped in a fixed position. For reading slopes, a slope board or clinometer arrangement is provided. A strap is attached to the back by a swivel connection and the board is held on the left hand or wrist and manipulated with the right hand.

Many different designs of sketching cases have been devised in an effort to provide an instrument that will do all of the work and replace skill on the part of the sketcher. In this latter purpose they generally fail, and the more elaborate designs are discarded by the sketcher as he acquires skill. The experienced sketcher prefers the simplest of means to aid him in his work.

The *Aneroid Barometer* is an instrument in which a light needle or pointer is actuated by change of atmospheric pressure. A thin metallic disc with circular corrugations forms a flexible cover for a sealed box from which the air is partially exhausted. The central point of the flexible cover is depressed more or less as the atmospheric pressure increases or decreases, and this slight motion is multiplied many times and transmitted to the pointer through a train of levers. Atmospheric pressure decreases with elevation above sea level, and the pointer may therefore be made to indicate, on a properly constructed scale, elevations above sea level.

The pressure varies also with changing atmospheric conditions, and the barometer will not, therefore, indicate absolute elevation unless the effect of these changes be eliminated. *Differences* of elevation may, however, be determined by the use of two barometers. One barometer remains stationary at a point whose elevation is known or assumed, and its observer records time and barometer readings at half hour intervals throughout the day. The other barometer, while at this initial station, is set at the same reading as the first, and then is carried by the surveyor or sketcher and read at the stations occupied during the day. The time of each reading is also recorded.

At the end of the day's work the two records are compared and the readings of the stationary barometer are subtracted from those of the surveyor's barometer taken at the same times. The results are differences of elevation, which, applied with proper sign to the elevation of the initial station, give the elevations of the corresponding stations occupied.

The foregoing method is applicable in position sketches and in rough surveys that cover small areas each day, but in road sketches, which may cover fifteen or twenty miles each day and which advance always into new territory, it is impracticable to compare the barometer readings of the sketcher with those of an observer at a fixed station. Moreover, elevations of stations are needed at once in order to finish the sketch up to the station occupied, and subsequent adjustment by comparison with a standard is impossible. Therefore, the differences of elevation shown by the readings of the sketcher's aneroid must be adopted, usually without correction. In settled weather, barometric changes take place very slowly, and the error from this source in passing from station to station will be small. The accumulated error during a day's work may be 100 feet or more, and this seems large, but if twenty miles have been covered, an error of 100 feet in elevation is only 1 in 1000. The hand level and the clinometer are subject to errors greater than this, and the aneroid need not be discarded as a sketching instrument on the charge of inaccuracy.

When unsettled weather causes rapid barometric changes, an effort may be made to apply corrections as follows: At 6 A. M. set the barometer to read the known or assumed elevation at the point occupied and record this reading, say 850 feet. At 7 o'clock read the barometer again, and start out on the day's work. Suppose this reading to be 862 feet, indicating a rise of 12 feet in one hour. Then a correction of — 12 feet must be applied to the 7 A. M. reading to get the original reading, 850. Assuming that this rate of change continues for two hours, the correction at 8 A. M. will be — 24 feet, and at 9 A. M. — 36 feet. Intermediate corrections may be interpolated. At 9 o'clock halt for half an hour, and take readings at 9 and 9:30, say 824 and 828 respectively, showing a rise of 4 feet in thirty minutes, or 8 feet per hour. Then at 10 o clock the correction will be — 44 feet, and at 11 o'clock — 52 feet. During a halt from 11 to 11:30

the barometer reads 810 feet without change, and the correction, —52 feet, is applied for the next two hours. The readings during the next halt at 1:00 and 1:30 P. M. are 866 and 864, showing a fall of 2 feet in 30 minutes, and giving corrections of —48 feet at 2 P. M., and —44 feet at 3 P. M.

At the 3:00 to 3:30 P. M. halt, the readings might be 916 and 910, giving corrections of —32 ft. at 4 P. M., and —20 ft. at 5:00 P. M. The resulting elevations of the stations occupied at the odd numbered hours would be—

7:00 A. M.	862 — 12 = 850.
9:00 A. M.	824 — 36 = 788.
11:00 A. M.	810 — 52 = 758.
1:00 P. M.	866 — 52 = 814.
3:00 P. M.	916 — 44 = 872.
5:00 P. M.	933 — 20 = 913.

For readings at intermediate stations, the time is noted and an interpolated correction is applied.

The record may be kept conveniently by plotting times, barometer readings, corrections, and corrected elevations on profile paper as shown in Plate 17. The horizontal scale is 1 inch = 2 hours and the vertical scale, 1 inch = 40 ft. The 6:00 and 7:00 A. M. readings are plotted, and the line through them is prolonged two hours to 9:00 A. M. The ordinate, for any time, from this line to the "level line," is the correction to be applied to a barometer reading at that time. Thus at 7:45 A. M. the barometer reads 847, which is plotted at a; but the correction at this time is a′ b′, and this, transferred by the dividers to a b, gives the point b at 826, which is the corrected elevation of the station.

At the first halt the 9:00 and 9:30 readings are plotted at c and d and this line is prolonged to 11:00 A. M. at e. Draw the parallel line c'e' prolonging the correction line to 11:00 A. M. In like manner the rate of change is determined at each halt, prolonged two hours, and transferred to the correction line by a parallel. In Plate 17 the corrections have been applied to the plotted readings at the odd numbered hours, and the line joining the corrected points and marked "profile" is a rough profile of the road traveled during the day, distance being expressed in time. If progress has been made at a uniform rate of say two miles per hour, a scale of miles may be added along the bottom line as shown. The horizontal portions representing the half-hour halts must be disregarded.

If corrections were not determined and applied, the elevations shown on the line marked "uncorrected profile" would have to be accepted.

The half-hour halts need not be time wasted, since they may be utilized in finishing the sketch or in locating distant points by triangulation.

The aneroid barometer is an expensive and delicate instrument, and must be handled and used with great care. The small pocket aneroid, about the size of a watch, can be read only to the nearest 10 feet, and is not satisfactory as a sketching or surveying instrument. The larger size, inclosed in a padded case and reading by careful estimation to the nearest 4 feet, is much to be preferred. Before reading an aneroid at any station, it should remain undisturbed for a few moments and then be tapped lightly to settle the needle. A magnifying glass should be used to get close readings. The aneroid must be shielded from the direct rays of the sun and must be protected from shocks.

Only the simplest and most obvious use of the aneroid as a sketching or reconnaissance instrument has been indicated, and the corrections that are necessary for obtaining good results in surveying have been disregarded. In mountain surveys the large aneroid or the mercurial barometer is a most valuable instrument. By simultaneous hourly readings for a day or more of two barometers at two stations, and by applying corrections for temperature and moisture, the difference of elevation of the two stations may be determined with considerable accuracy. The use of the barometer in hypsometric surveys is a special study that cannot be considered here.

SCALES OF MAP DISTANCES.

The standard system of scales and vertical intervals adopted for military maps, as explained in Chapters I and II, gives for each degree of slope a constant map distance between contours for all maps that conform to the system, no matter what the scale. For a slope of 1° the map distance is 0.65 of an inch. For any other slope up to 20° divide 0.65 by the degrees of slope, and the result is the map distance for that slope expressed in inches.

The instruments used in sketching will read vertical angles or slopes only to the nearest half or quarter degree, and scales may be prepared on cardboard giving map distances for each quarter degree up to 3°, for each half degree up to 6°, for each degree up to 12°, and for each two degrees up to 16°. All of these scales may be placed on the edges of one card as shown in Plate 18. The scales for 9°, 10°, 11° and 12° may be subdivided by eye to give map distances for 18°, 20°, 22° and 24° respectively, although these steeper slopes are seldom encountered.

When the slope of the ground along any line is read with the clinometer or slope board, the scale for that slope is applied along the corresponding line on the sketch, and, beginning at the station occupied, contour points are dotted along the line at the marks of the scale. By counting one vertical interval for each space, the elevation of any point on the line may be determined.

THE SKETCHING RULER.

The ruler used in sketching serves several purposes. If the plane table method be used, as with the sketching case or oriented field drawing board, the ruler serves as an alidade, to be pivoted at the point representing the station occupied, and sighted at the "point observed." It serves next as a straight edge for drawing courses and side shots, and finally, with divisions marked on its edge representing to proper scale the units of measure used in sketching, it serves to lay off distances.

A convenient form of ruler for these purposes is shown in Plate 19. It is triangular in cross section and is five inches long. The three faces of the ruler give six edges on which scales may be marked. By placing different sets of scales on each half of the ruler, twelve scales are provided, as follows:

1. Yards.
2. Man's pace or stride.
3. Horse's step or stride at walk.

4. Horse's step or stride at trot.
5. Horse's time at walk.
6. Horse's time at trot.

And scales of map distances in the following sets:

7. ½°, 1°, 2°, 4°, 8°, 16°.
8. 1¾°, 3½°, 7°, 14°.
9. ¾°, 1½°, 3°, 6°, 12°.
10. 2¾°, 5½°, 11°.
11. 1¼°, 2½°, 5°, 10°.
12. 2¼°, 4½°, 9°.

These are the same map distance scales as those shown in Plate 18, and when this ruler is used the card scales will not be necessary.

The ends of the ruler are capped with pieces of sheet rubber, trimmed to project slightly beyond the faces. These edges support the ruler at the ends only, and cling to the paper with adhesion sufficient to prevent displacement when drawing a pencil line at the edge. To assist in this purpose and to prevent displacement by wind, the ruler is weighted with lead in a hole bored lengthwise through the axis. A small ring at one end may be used to attach the ruler by a string to a button-hole.

The scales of paces, steps and times must be constructed for the individual sketcher and his horse. They may be marked on slips of paper and glued to the edges of the ruler. The face of the ruler seen in Plate 19 shows two trotting scales for a horse that trots a mile in 7½ minutes with 45-inch steps, at a scale of 3 inches to the mile, and the map distance scales correspond to 20 feet V. I. If sketching at a scale of six inches to the mile, divide the numbers on the pace, step and time scales by two and use a V. I.

of 10 feet. At one inch to the mile, multiply the numbers by three and use a V. I of 60 feet. If strides are counted instead of paces or steps, divide the numbers on the pace or step scales by two.

With this ruler properly marked, the sketcher will be prepared to work mounted or on foot, using paces, steps, strides or time, walking or trotting, at scales of 1 inch, 3 inches or 6 inches to the mile; to space contour points for any slope at vertical intervals of 60, 20 or 10 feet respectively, and to change from one method or one scale to another by turning the corresponding edge of the ruler to the paper.

Scales for different lengths of paces, steps or strides, and for different rates of travel at walk or trot are given on Plates 20, 21 and 22. Any desired scale may be copied from these plates by placing the edge of a strip of paper on the proper line or by interpolating between the lines.

METHODS OF SKETCHING.

Different methods of sketching depend on the different sets of instruments that may be used to maintain general control in directions, distances and elevations. In all methods of sketching the chief reliance is placed on the ability of the sketcher to draw free-hand on the paper what he sees on the ground.

THE NOTEBOOK METHOD.

In this method a record is made in a notebook of all the observations made with the hand instruments in the field, and on the right hand pages of the notebook free-hand sketches are made, showing the ground covered by the notes on the corresponding left hand pages. After the day's field work is finished, the notes are plotted on a sheet of drawing paper, and the "filling in" is done by copying and adjusting the free-hand sketches of the notebook.

The outfit required for the field work consists of a box or prismatic compass, a clinometer, a stop watch, an aneroid barometer, notebook, pencils, etc. The sketching may be done on foot, on horseback, or in a wheeled vehicle. With the latter an odometer is usually attached to one of the front wheels to measure distances along the trail.

The program of operations is as follows:

At the initial station, read the aneroid, and if its rate of change has been determined by previous readings apply the correction that corresponds with the time of this reading, or plot aneroid readings, corrections, and corrected elevations on a piece of profile paper, as previously explained. With the compass, read bearings along the trail to the selected station in advance and to the most important critical points in the vicinity. With the clinometer, read vertical angles to the selected station and critical points, and read also the slope of the ground in the same directions, estimating the distances to these critical points and also the distances over which the observed ground slopes remain uniform. Record these readings on the left hand page of the note book and make a free-hand sketch on the right hand page showing roughly, but approximately to scale, the "points observed" and the features located by them. Observe only the points and slopes that are absolutely necessary for general control, and rely on the free-hand sketch for all details.

The record is made in form similar to that of a compass survey, with the following headings and columns:

Date......................

Road Sketch from.................... to......................

Made by .... at trot, 1 mi. in 8 minutes.

PRISMATIC COMPASS. (*Plotting Diagram.*)

1	2	3	4	5	6	7	8	9
At Sta.	Point Obsr'd.	Dist.	Bear'g.	Vert. Ang.	Time.	Aner-oid.	Elev.	Remarks.
1					7:45	847	826	At Cross Roads.
	Top of Hill.	360 y	225°	+3¼°				
	Slope.	140 y	225°	+1¾°				
	etc.			etc.				
	Sta. 2.	2 m 20 s	13°	+¼°				
1m 25s	House.	80 y	R					
1m 48s	Down Stream.		97°	−½°				At Bridge.
	Up Stream.		260°	+¾°				At Bridge.
	etc.							etc.
2					8:05	914	890	
	Water Shed.	250 y	278°	+2°				
	etc.							etc.

In the first column are the numbers of the stations occupied. Intermediate halting points are recorded in this column by distances along the trail expressed in stop watch time from the last station. At every halt the stop watch must be stopped at the instant of halting and started at the instant of resuming the march. At the station halts, after its reading is recorded, it is set back to zero. It will then indicate always the distance in time from the last station. The third column records distances, expressed in yards for estimated side shots, and in time for the courses between stations. In the fourth column the bearings of lines are entered, and the letters R and L are used to denote rectangular offsets to the right and left respectively of the trail. Thus at 1 min. 25 sec. from Sta. 1, a house stands 80 yards to the right of the trail. Column 5 contains vertical angles to points observed and slope of ground. When the slope of ground is recorded it is so noted in column 2.

Columns 6, 7 and 8 contain the record for the aneroid. When the aneroid is not used, or when its record is plotted on profile paper, these columns are omitted.

The column of "Remarks" contains such descriptions of stations or of points observed as may be necessary.

The sketching on the right hand pages is facilitated by having a protractor circle printed on these pages as in the Engineer Field Notebook. Knowing the general direction of the road or trail to be covered, mark that direction at the top of the protractor in terms of its compass bearing and number the other 10 degree marks with their corresponding compass bearings for the compass used. Then, when facing forward along the trail, directions transferred by eye from the protractor to the station points on the sketch will be approximately oriented with the corresponding directions on the ground. The scale of the sketch must be carried in the mind so that distances may be plotted by estimation. Beginners should refer frequently to constructed scales until a correct mental standard is fixed. For spacing contours, the standard map distances for different slopes soon become fixed in the mind and estimates will be fairly accurate.

Alternate pairs of pages between those for record and sketch are used for notes describing roadway, bridges, crops, forests, villages, mills, shops, camping places, water supply, defiles, etc., with references by letter or number to points correspondingly lettered or numbered on the sketch.

The record and sketch should include all details that can be drawn to scale to a distance of 300 or 400 yards on both sides of the trail. Villages and prominent natural features should be included to a distance of 800 or 900 yards, their location being determined by intersections from at least two points.

A method often used for recording notes of a sketch is to begin at the bottom of the page and work toward the top, placing bearings and distances of the main traverse in a center column, and offsets right and left in columns at the right and left respectively of the center column. This method has no advantages over the method given above, and is simply an attempt to keep the notes as well as the sketch oriented with the forward progress of the work. The notes are still nothing but notes, and must be plotted after the field work is finished. The customary method of writing from the top of the page down is more natural and less confusing.

Plotting the Sketch.—When the day's field work is completed, the sketch is plotted and filled in like an ordinary compass survey. The main traverse along the trail is first plotted and marked with the elevations of its stations. Then the side shots are plotted to locate important control or critical points, and these are marked with their elevations. Contour points are dotted on all lines whose slopes have been determined. If these lines have been well selected, they will include all the principal control lines, especially watersheds and water courses. Then, by copying the freehand sketches of the notebook, fill in the details and make them conform to the control points determined by the main traverse, side shots and slope lines.

For these purposes there will be required a small protractor marked to correspond with the compass readings, a ruler marked on its edges with a scale of yards and a scale of steps or time, whichever was used in measuring distances, and a set of map distance scales on card or ruler. The use of these instruments has been explained in previous pages.

If the aneroid barometer has not been used, elevations must be carried forward from station to station by means of the recorded slopes and distances, using the map distance scales to determine differences of elevation.

Thus, Plate 23 shows three courses of a traverse made at a trot of 8 minutes to the mile, and the recorded slopes are $+2\frac{1}{2}°$, $+6°$ and $+3\frac{1}{2}°$, respectively. Use the protractor and the 8 minute scale, 1:21120, to plot the courses, and then apply to the first course the map distance scale for $2\frac{1}{2}°$. The known or assumed elevation at the first station is 925, the V. I. is 20 feet, and the next contour point will be at 940. Therefore, place the quarter point of the first division at this station and dot in the 940, 960, 980 and 1000 contour points, the last falling at the second station and giving its elevation. On the second course apply the 6° scale and there are found to be seven and one-half of its divisions, giving 1150 as the elevation of the third station. On the third course apply the $3\frac{1}{2}°$ scale, beginning at the middle point of the first division, and point off the map distances. There will be six points, and, estimating by eye, four-fifths of another division, which adds 16 feet, and gives 1276 as the elevation of the last station. In like manner, elevations may be carried from station to station throughout the day's traverse.

By plotting the slopes on side shots from the station points, additional contour points are determined and contours may be drawn, as shown in the figure.

When the aneroid barometer is used to determine the elevations of stations, no attempt is made to carry a continuous chain of slopes throughout the traverse, but at each station slopes are read on the principal control lines, and in plotting the work the station point is used as an origin for locating and sketching surrounding features, which are made to join those determined from adjacent stations.

If the sketch is to be combined with other sketches of parallel roads to form a map of the region covered, it may be finished in black pencil, but if it is to be used in its original form, the following conventional colors are added with crayon pencils to bring out the different details distinctly. Trace the roads with yellow; stream lines, ponds and lakes with blue; contours with red; and lay on flat tints of green for forest masses, and of brown for cultivated fields. The fences, houses, bridges, cuts and embankments, railroads and other artificial features, remain in black.

Every sketch must contain a title, a scale, a magnetic meridian, the date and the name of the sketcher.

The descriptive notes kept on alternate pairs of pages in the notebook are used as the basis of a report which is submitted with the sketch. The reference letters and numbers should appear on the finished sketch.

SKETCHING CASE METHOD.

The sketching case is simply a small plane table. It is oriented by means of the attached compass, and its ruler serves as the alidade. For reading vertical angles or slopes it becomes a slope board and is held in a vertical plane with the ruler swinging freely on its pivot. Sight along the top line of the board at the "point observed" and read the angular scale on the lower edge at the point cut by the edge of the ruler. The clinometer is to be preferred for reading vertical angles, and may be used in connection with the sketching case. The aneroid barometer is also useful for obtaining elevations of principal stations.

In this description it will be assumed that the sketching case alone is used, but it should be understood that the clinometer and aneroid are useful adjuncts.

At the initial station hold the board squarely in front of the body and face in the general direction which the sketch is to follow. Loosen the needle and turn the bars or lines of the compass cover to a position parallel with the needle. Draw on the paper parallel with the needle a magnetic meridian. Assume a point on the paper and mark it as the initial station. Select the critical points that must be determined, and for each of them face squarely toward it, orient the board by bringing the bars or lines parallel with the needle, place the edge of the ruler on the station point and sight it at the "point observed," draw a line along the edge of the ruler, estimate the distance in yards and plot it with the scale of yards.

Having thus plotted the selected points, use the slope board (or clinometer) and read the vertical angles or ground slopes toward the same points, apply the proper map distance scales and point off the contour points. Draw the contours thus determined, shaping them by constant observation of the ground surface, and sketch in all the surrounding details that can be drawn to scale. Lastly, face along the trail toward the next selected station and plot its direction and slope.

Walk or ride to the next station, halting when necessary to sketch in adjacent features, and at the new station plot its position by the measured distance in paces or time along the trail. Determine its elevation by map distances from the first station (or by the aneroid), and proceed here, in the same manner as at the first station, to locate and sketch surrounding features and details, and continue in like manner from station to station throughout the day's march, finishing the sketch as it progresses.

With practice and experience many of the foregoing operations may be curtailed or omitted, and more reliance be placed on the skill of eye and hand in sketching what is seen. The traverse along the trail should, however, always be determined by actual measurement of directions, distances and slopes, and not by estimation.

A fifteen-mile road sketch will, at 3 inches to the mile, cover 45 inches of paper, and if the road varies in direction, or if the original orientation has been wrong, the sketch will run off the edge of the 6 inch strip, which is rolled back as the sketch advances. When the main traverse approaches the edge of the paper with future promise of running off, terminate the completed portion of the sketch by drawing a line across the paper, and begin anew, assuming a central position on the paper for the station occupied. Correct the orientation so that it shall conform to the new direction of the trail, and draw the corresponding magnetic meridian. Several such changes may be necessary in the course of the day's work, dividing the sketch into as many separate sections. When the sketch is ended, cut the sections apart and lap the ends of adjacent sections, making the common station points coincide by means of a pin stuck through both. Turn the forward section so that its meridian line shall be parallel with that of the previous section, and cut through both sections on a line that will save as much as possible of the sketch. Fasten the sections together in this position by gumming a strip of paper across the joint on the back. The several sections will then be properly oriented with respect to each other and will form a connected map of the strip of country traversed by the trail.

If the route followed makes a circuit and returns to the starting point, the sketch must be made to close. Pin the sections of the sketch to a board or table with pins stuck through the common station points of successive sections and make a closed circuit. Then adjust and shift the pin joints so that the meridian lines of all the sections shall be as nearly as possible parallel. Cut through at the joints and fasten the sections together by pasting them on a single large sheet of paper. If portions of the sketch must be trimmed off in making the joints, they must be copied in the corresponding blank spaces on the adjacent section.

The sketch is finished and colored in the manner described for the "notebook sketch," not omitting meridian line, scale, title, date, and name of sketcher.

While making the sketch a notebook is used for notes that will be needed in preparing the usual report that is submitted with the sketch.

FIELD DRAWING BOARD METHOD.

This method is well adapted to sketching on foot, and is suitable for either road or position sketching. the drawing board is 12" x 15" and ½ inch thick, made of white pine and reinforced at the ends. On the back of the board is a slope board arrangement, consisting of a small plumb line and bob attached at the middle of the upper edge, and a scale of degrees near the bottom edge, with its zero at the middle point. Sight the top edge at the point observed and when the plumb line comes to rest hold it in place with the finger and read the scale at the point cut by the line. This is a rough method at best and cannot be used in a high wind. The clinometer should be used when available.

A cord attached to the left front and right rear corners of the board and passing over the left shoulder and under the right arm supports the board horizontally in front of the body when sketching, slings it under the right arm while pacing, and does not interfere with its use as a slope board.

A sheet of ruled paper is attached to the face of the board by thumb tacks.

The outfit consists of the drawing board, a box or prismatic compass, a small rectangular protractor, scales of paces and of map distances, notebook, pencils, etc. The clinometer and aneroid barometer are useful adjuncts in place of the slope board attachment, and it is understood that they will be used when available and when the best results are desired. A passometer or pace tally is convenient for keeping the count of paces. Ruler, protractor and scales may be combined in one instrument. The best combination is a stiff card, marked on one side with a protractor covering three of its edges and a scale of yards and scale of paces on the two halves of the fourth edge. The other side contains the scales of map distances as shown in Plate 18. Otherwise, the ruler shown in Plate 19 may be used in addition to the protractor.

At the initial station sight with the compass at selected critical points and at the next forward station. At the point assumed on the paper for "station occupied," plot, with the protractor, these compass readings as each is determined, and lay off the estimated distances with the scale of yards. Sight with the slope board (or clinometer) at the same points and read the vertical angle to each. Apply the proper scale of map distances on each line and dot the contour points. Draw the contours and fill in the surrounding details by free-hand sketching, locating them with reference to the plotted points and lines. While sketching, the board may be roughly oriented by pointing one of the plotted lines at the corresponding point on the ground.

Pace to the next station, counting paces and halting when necessary to sketch in adjacent features. The halting point is plotted by laying off its distance in paces from the station, and details, right or left, are located by estimated rectangular offsets. Arriving at the new station, plot the paced distance and determine elevation by map distances (or by the aneroid), and here proceed as at the first station.

The lines on the ruled paper are assumed to be meridian lines, and are so used in setting the protractor with its center at the station point and its meridian edge parallel to the adjacent meridian line. Without the ruled lines it would be necessary to draw a meridian line through every station. The letters N. and S. are marked at the ends of one meridian line in order that no mistake shall be made in setting the protractor.

Two or three strips or sections of the sketch may be run across the sheet of paper, each section beginning anew at the near edge when the sketch has run off at the far edge. When the first sheet is filled a new one is tacked on.

The sections are cut apart and joined end to end with parallel meridians and matched station points in the manner described for the sketching case method, and the sketch and report are finished in like manner thereto.

SKETCHING WITH IMPROVISED INSTRUMENTS.

An experienced sketcher who has acquired correct mental standards for estimating directions, distances and slopes, and for representing on paper to proper scale what he sees on the ground, needs only a pencil and a piece of paper to produce a fair road or position sketch. The paper, preferably in the form of a pad or block, is oriented by backsighting on a plotted line, and so held while new directions are added by pencil strokes toward the new points, a glance toward each "point observed" being sufficient to determine its direction. If the sketch runs generally towards a well defined point on the distant horizon, such as a mountain peak, the sketching board or pad may be oriented by sighting one of its edges toward the distant point, and whenever that point is visible the same orientation may be repeated. Estimated distances in yards and measured distances in paces or time are laid off, and surrounding features and details are sketched in by means of the "memory scales" that have been acquired by previous use of the standard scales of distances and contours. If a standard set of scales has not been used, these mental standards cannot be acquired with any degree of accuracy.

If greater accuracy be desired, as in a position sketch, or if the sketcher has not sufficient experience to rely on estimated distances and mental scales, a simple plane table outfit may be devised and prepared with such material as can usually be found.

The lid of a cracker box or other small packing case will serve as a drawing board. Cut three sticks, from straight brushwood or branches, about four feet long, lay them side by side, and about six inches from one end lash them together with over and under turns of cord or twine tightened with frapping turns in the intervals. Stand this up and spread the legs and it becomes a tripod. The board rests on the projecting top ends and may be leveled by adjusting the legs and sighting in the plane of the board at a distant horizon line.

For constructing a set of scales, a scale of inches is the first requisite. The sketcher should know some dimension on the hand, as the length of a finger or the distance between certain lines of the palm in inches. This distance may then be marked on a strip of paper which is folded into a number of equal folds, equal to the known number of inches, and a scale of inches is the result.

The size of the glove is the number of inches around the hand at the knuckles. A strip of paper may be wrapped around the hand as if measuring for gloves; if the size of the glove is No. 8, the strip that encircles the hand is 8 inches long; folded, it is 4 inches; another fold gives 2 inches, and a third fold, 1 inch; and a scale of inches is provided. One of the inch divisions may be subdivided by eye or by construction into ten equal parts to give tenths of inches, and these may be subdivided by estimation to the nearest hundredth of an inch. With this scale of inches and tenths, any desired working scales of paces, time, or yards may be constructed in the usual manner.

Remembering that 0.65, or 6½ tenths of an inch, is the map distance between contours for 1 degree, lay off this distance with the scale of tenths successively on the edge of a card for the 1° scale, halve these divisions for 2°, halve again for 4°, and so on for 8° and 16°.

Dividing 0.65 inches by 1½ gives 4⅓ tenths as the map distance for 1½°; and this may be successively halved for the 3°, 6° and 12° scales.

In like manner all of the scales shown in Plate 18 may be constructed and marked on the edges of a visiting card, or other rectangle of stiff paper.

Vertical angles or slopes may be read by using the board as a slope board. Select one edge as a sighting edge, and near the middle of this edge on the back of the board attach a thread by driving a small wooden wedge into the board. A bullet or key will do for a plumb bob. Through the point of attachment draw a line perpendicular to the sighting edge. The zero of the angular scale must be on this line, and this angular scale must be constructed on both sides of the zero line.

With the thread as a radius, strike an arc with a radius of 5.73 (or 5¾) inches. Beginning at the zero line, lay off consecutive one-inch chords on this arc in both directions, and divide each chord into tenths. The divisions so marked will be degrees, because the tangent of 1 degree is $1 \div 57.3$, and in this construction tenths of an inch have been laid off approximately perpendicular to a radius of 5.73 inches. The degree lines may be prolonged to the bottom edge of the board and there marked by stretching the thread through each of the points.

A ruler may be whittled out of any small piece of straight grained wood. It may be smoothed and straightened by rubbing its faces and edges on a board covered with fine sand. The triangular form is best, as this gives a good sighting edge when it is used as an alidade. The scales that have been constructed may be marked on its edges. A piece of paper is fastened to the board by driving small wooden wedges at the corners through the paper into the board. Pencil and eraser complete the outfit.

At the initial station set up the tripod, level the board, and assume a point on the paper to represent "station occupied." Pivot the ruler's edge at this point and sight it at surrounding objects and critical points, drawing the corresponding direction lines, including one to a point selected for the next station. Estimate distances on side shots and lay them off with the scale of yards, or pace these distances, if estimates cannot be relied upon, and lay off with scale of paces. Then pick up the board, and using it as a slope board, read the vertical angles or slopes on the same lines, and with the map distance scales dot the contour points from an assumed elevation for the station occupied. Draw the contours, shaping them by observation of the ground forms, and sketch in surrounding details.

Place a mark at this station, such as a stake, stone, branch or bunch of grass and weeds—anything that can be seen from the next station—and pace the distance to the next station. Here set up the tripod, level the board and plot the distance in paces from the first station. Find the elevation by map distances and fraction thereof. Orient the board by backsighting at the previous station with the ruler's edge on the line that represents the course. Then proceed at this station, as at the first station, to locate and sketch surrounding features and details. Continue these operations at successive stations throughout the day's traverse. At intermediate halts between stations, the board may be held in the hand and approximately oriented by sighting at the back or forward station. Sometimes a point on the distant horizon may be selected which will assist in orienting the board when a previous station is lost sight of.

An approximate meridian line may be drawn as follows. Place the watch on the oriented board and point the hour hand toward the sun. A line which bisects the angle between the hour hand and the XII mark will then point approximately to the south, and such a line may be drawn on the paper as a meridian line. For this purpose the watch should show local time.

Otherwise, if camp is made near the end of the day's traverse and if the sketch is not needed at once, leave the board oriented at the last station, and at dark sight the ruler at the North Star and draw the meridian.

POSITION AND OUTPOST SKETCHES.

Any of the outfits and methods heretofore described may be used for any kind of sketching—route, road, or position, on foot or on horseback—but certain outfits and methods are better suited than others for certain purposes. A position sketch is usually made at a scale of 6 inches = 1 mile with V. I. = 10 feet, and includes much more detail than does either the route or road sketch. The area to be covered is correspondingly smaller, or for an extended position, it is divided into areas of from one to two square miles each, and these are assigned to the members of a party of sketchers, whose sketches are subsequently combined into a single connected map. The sketch does not necessarily or usually follow a road, but runs across country on lines that give the best control and best view. It is made on foot, and the field drawing board or a light plane table outfit are most suitable for the purpose, better than the sketching case or the notebook.

A "position" usually extends along high ground that commands the slopes and valleys to the front, and the sketch should include the position and the ground in front to a distance of at least two miles.

Run a careful base line traverse for general control along the position, leaving the stations plainly marked on the ground, and determining their elevations with the aneroid barometer. Then run secondary traverses in loops to the front, beginning at one of the main traverse stations and closing on another station. Each loop should inclose a strip about half a mile wide and extending about two miles to the front. The inclosed space not reached from the stations of the loop traverse may readily be filled in by eye. Each strip must be adjusted and connected with adjacent strips, so that the whole shall form a connected and consistent contoured map covering the position, and showing all essential details.

By assigning such strips to the individual members of a party of skilled sketchers, an extended position may be covered in a day, and the sketches may be combined and printed on blue print or bromide paper and be ready for issue by dark of the same day.

Outpost Sketch.—The purposes of an outpost sketch are similar to those of a position sketch, since it is intended to furnish a detailed map which will serve for the posting of troops, locating field works and batteries and determining ranges, in case an engagement is expected at the position occupied.

The method of sketching must, however, sometimes be modified on account of the presence of an enemy which will prevent the running of traverses to the front and will confine the operations of the sketcher to the line of outposts. If the ground to the front is open it may be covered by triangulation, that is, by intersections and vertical angles from selected stations. For this method a light plane table outfit is most suitable, and the "improvised outfit" described under that head will serve. The aneroid barometer and clinometer will, of course, give better vertical control and should be used when available.

Select stations on the "line of observation" which will give good views over all the foreground without unnecessarily exposing the observer. Connect these stations by a careful traverse, which may be run under cover in rear of the line of stations, and reach them by offsets or short branches. Occupy these principal stations in succession, and at each station orient the board and draw direction lines by sighting the ruler at prominent objects and important critical points in the foreground. For the nearer points, adjacent stations will give good intersections, but for distant points, widely separated stations will give a longer base and better intersections.

When a distant point is located by intersection from two stations, read the vertical angle to that point with slope board or clinometer and determine its elevation by an application of the proper scale of map distances, counting from elevation of "station occupied" as determined by the main traverse.

Write the elevation at each determined point. Having located a sufficient number of control points, interpolate contour points on the lines joining them and sketch in the contours. If the critical points have been well selected, the sketching of intermediate features and details should be easily accomplished.

When the country is generally broken and wooded, the foregoing method is not applicable, and it will be practicable only to run short traverses from the main traverse to the front, on roads or paths under cover of patrols. For this purpose the field drawing board or the sketching case is suitable, and the sketches thus made may be combined to produce a fair map of the position and the foreground.

Since the outpost sketch is intended to serve the same general purpose as a position sketch, it is usually made to the same scale, namely, 6 inches to the mile with V. I. of 10 feet. For an extended position, covering perhaps 10 to 20 miles of front, a scale of 3 inches to the mile with V. I. = 20 feet, may be used, and the work would necessarily be divided among the members of a party of sketchers, to be subsequently combined into a continuous map. The same accuracy and attention to detail is not expected in an outpost sketch, since the ground cannot usually be traversed and thoroughly examined as in the position sketch.

REMARKS ON SKETCHING.

For many purposes a carefully made sketch map is just as good as one made by an accurate instrumental survey, and since a road sketch will cover in a day the ground that it would require a week to survey, the value of sketching may be appreciated, especially in connection with military operations, where a map, to be of use, must be immediately available.

For some civil purposes, such as the preliminary examination of a route for a road or railroad, or in exploratory expeditions in new countries, the methods of sketching are equally applicable and valuable.

It need not be assumed that the rough and hasty methods adopted in sketching will give only rough and approximate outlines of the ground covered. The degree of accuracy that may be attained after long practice and careful training of eye and hand is surprising to one not familiar with the results of such training. Maps produced by these methods and by experienced sketchers need no apology on the ground of haste or inaccurate instruments, since they will serve every purpose that an ordinary map is expected to fulfill, and for military purposes will include many details and features not usually shown on the civil maps of the region.

Sketching parties, advancing on parallel roads under cover of the cavalry screen, and sending individual sketchers in turn to cover the cross roads, will in one day turn in sketches which, being combined, adjusted, traced and printed, will furnish for issue before midnight of the same day, a map covering the entire front of the army, perhaps twenty or thirty miles from right to left, and extending a day's march, say fifteen miles, to the front. This map should show all of the roads in the region covered, with details of topography on a strip one-half mile wide along each road, and the most prominent features in the intervening areas. The accompanying reports will give all the additional information which may be useful in ordering the day's march, selecting camping places, and in making proper tactical disposition of troops and trains. Similar maps are turned out for each day's advance of the cavalry screen. When necessary, the intervening areas not covered from the main roads may be filled in by other parties of sketchers, and a new and complete edition of the maps may be issued.

Upon contact with the enemy, the sketchers are set to work making detailed position sketches, which should cover not only selected positions, but as much of the ground towards the enemy as time will permit. These are also printed and issued for each day's work, but may subsequently be combined and printed as one map (perhaps in several sheets) of the front of operations.

For the service which has been outlined above, it is evident that a well trained body of sketchers is necessary and that they must be organized and controlled under one chief, who will see that all of the ground is covered, that none of it is covered more than once, and that all the sketches are turned in at the appointed place and time, to be combined, printed, and issued at the earliest possible moment.

Every individual road sketch or position sketch should be made with the idea that it is to be combined with adjacent sketches. To this end, the standard scale and vertical interval must be adhered to; the terminal stations must be well defined points, easily identified, and carefully described in the report; cross and branch roads should be marked with the name of, and distance to, the nearest town or village; local names of villages, streams, heights, railway stations, postoffices, etc., should be ascertained and printed at appropriate places on the sketch, and every piece of paper that contains a portion of sketch should contain also a magnetic meridian, a scale, the date, and the name of the sketcher. If these points are carefully attended to by each sketcher, the chief of the party will have no difficulty in combining the several sketches that are turned in to him.

Only the regular methods and operations of sketching have been described in foregoing pages. There are numerous checks, tricks, shortcuts and aids that are applicable under various circumstances and that will occur to the sketcher as he acquires experience. A few may be mentioned to indicate their general character.

Horizontal sights with clinometer or slope board from a given station will locate a number of points of equal elevation which may be used as check points in subsequent work.

By sweeping a hillside with a horizontal line of sight, a contour may be traced on the ground to serve as a guide in tracing the contour on the sketch.

A line of sight to a distant horizon is very nearly horizontal.

When two or more prominent objects come into range from a point on the trail, halt and plot the line through those objects. Subsequent intersecting sights on the same objects will determine their positions.

When coming on line with a distant stretch of road or river, plot the line and use it as a check on subsequent location of the same stretch in case the trail runs to it. If adjacent reaches of river are thus plotted and joined by curves, that portion of the river will be fully determined, although the trail may not run near it.

In road sketching it is not necessary always to follow the road. A road often runs between high banks or hedges that obstruct the view, and it may be better to traverse a line on high ground near the road, which will give better control and more extended view. The road may be readily located by offsets from the traverse.

Three or more plotted sights to the same object from different points of the trail should intersect in a common point.

A parallel road may sometimes be discovered by noting the passage of a wagon or horseman along it. If several positions of the wagon be estimated and plotted, a portion of the parallel road may be sketched. Similarly, the passing of a locomotive in the distance will disclose the location of a railroad that might not otherwise be discovered.

In some regions an outcrop of ledge rock maintains the same elevation for many miles, and will serve as a check on elevations.

Similar checks and aids will occur to the sketcher as his work progresses, and he should make use of every device that will expedite his work and increase its accuracy. Nothing, however, can take the place of the skill that is acquired only by practice.

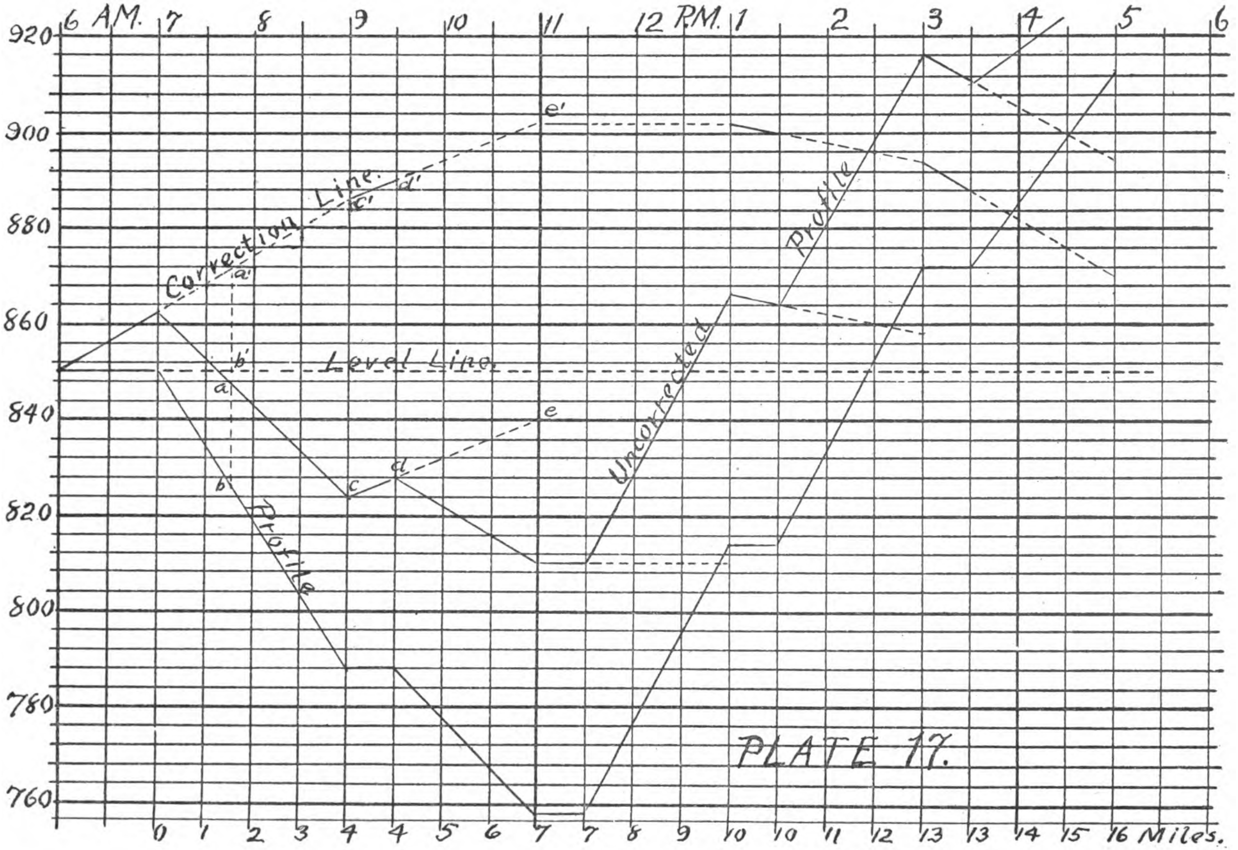
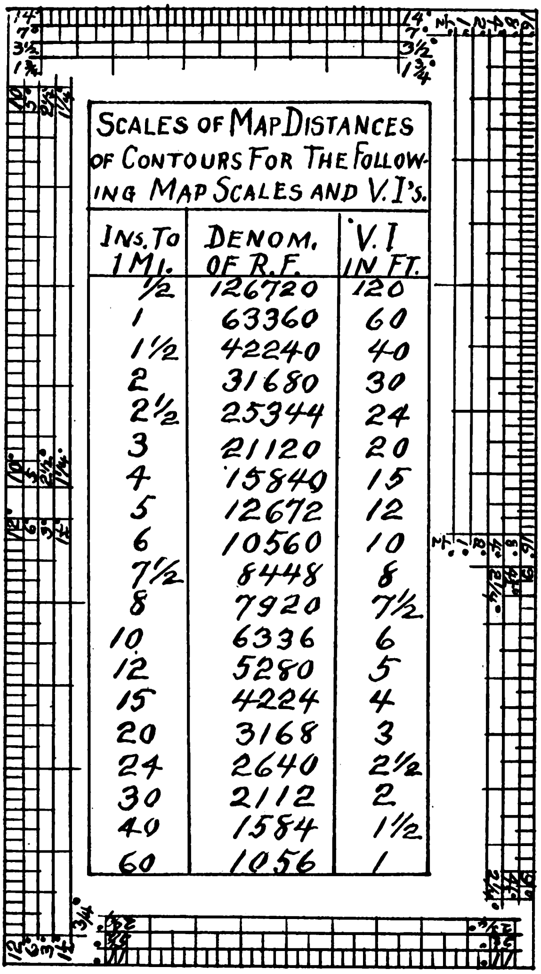

6 A.M.
7
8
9
10
11
12 P.M.
1
2
3
4
5
6
920
900
880
860
840
820
800
780
760
Correction Line.
Level Line.
Profile
Uncorrected
Profile
a
a'
b
b'
c
c'
d
d'
e
e'
0
1
2
3
4
4
5
6
7
7
8
9
10
10
11
12
13
13
14
15
16 Miles.

PLATE 17.



SCALES OF MAP DISTANCES OF CONTOURS FOR THE FOLLOWING MAP SCALES AND V. I's.

INS. TO 1 MI.	DENOM. OF R. F.	V. I IN FT.
1/2	126720	120
1	63360	60
1 1/2	42240	40
2	31680	30
2 1/2	25344	24
3	21120	20
4	15840	15
5	12672	12
6	10560	10
7 1/2	8448	8
8	7920	7 1/2
10	6336	6
12	5280	5
15	4224	4
20	3168	3
24	2640	2 1/2
30	2112	2
40	1584	1 1/2
60	1056	1

PLATE 18.

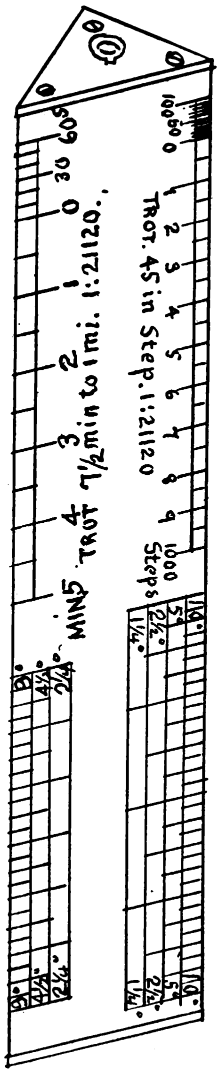

TROT. 45 in Step. 1:21120
1000 Steps
TROT 7½ min to 1 mi. 1:21120
MINS

PLATE 19.

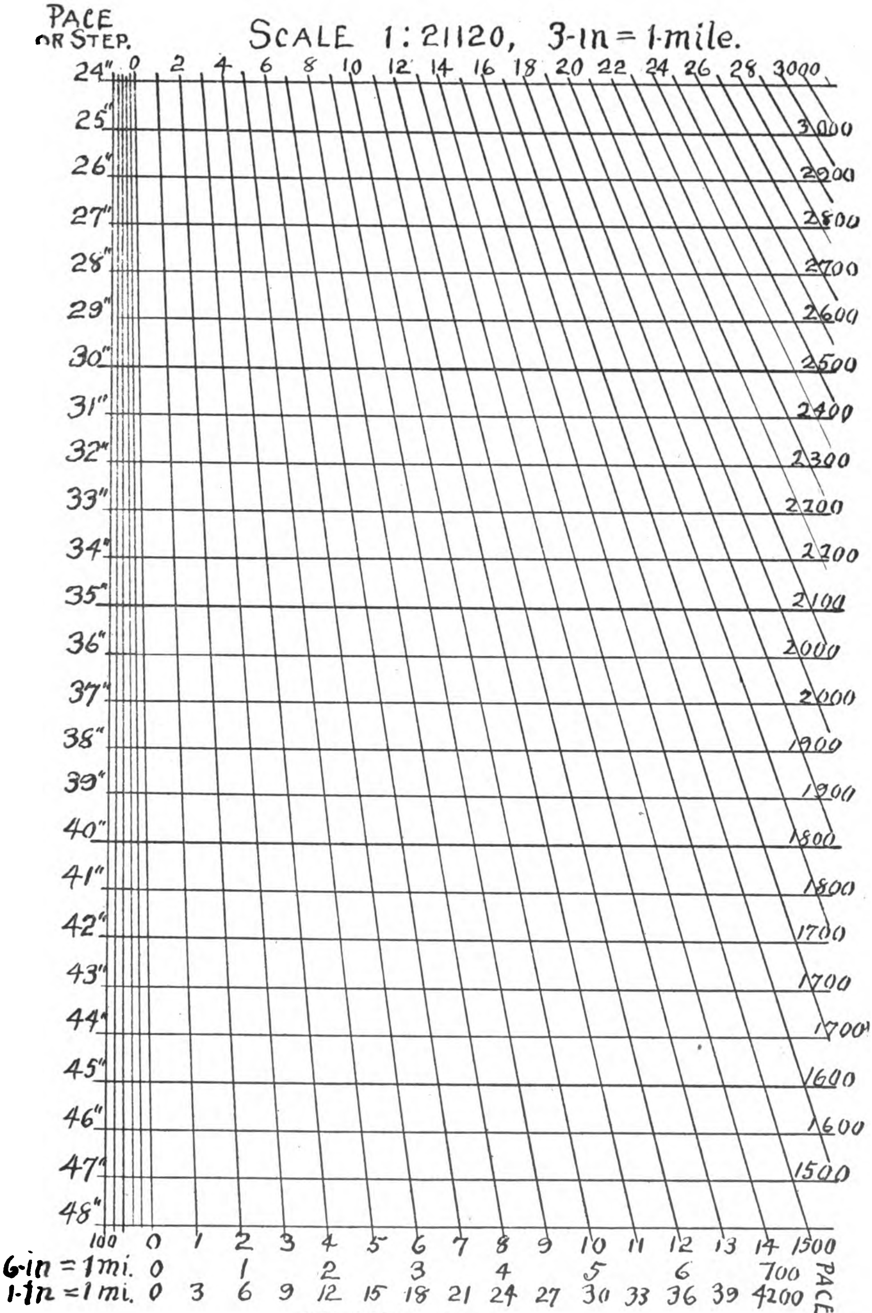

PACE OR STEP.
SCALE 1:21120, 3-in = 1-mile.
24" 25" 26" 27" 28" 29" 30" 31" 32" 33" 34" 35" 36" 37" 38" 39" 40" 41" 42" 43" 44" 45" 46" 47" 48"
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 3000
100 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 1500
6-in = 1 mi. 0 1 2 3 4 5 6 700
1-in = 1 mi. 0 3 6 9 12 15 18 21 24 27 30 33 36 39 4200
PACES.

PLATE 20.

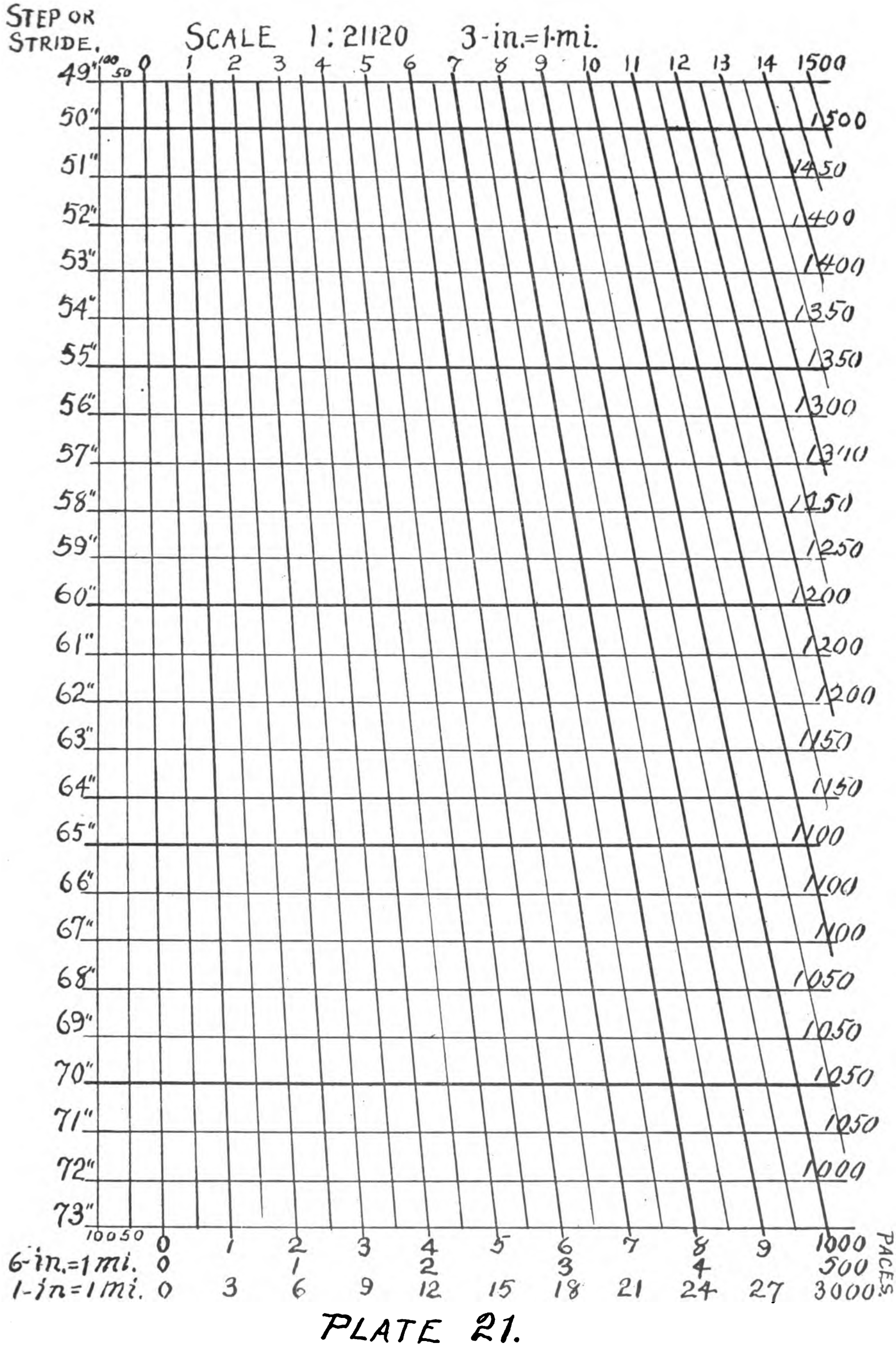
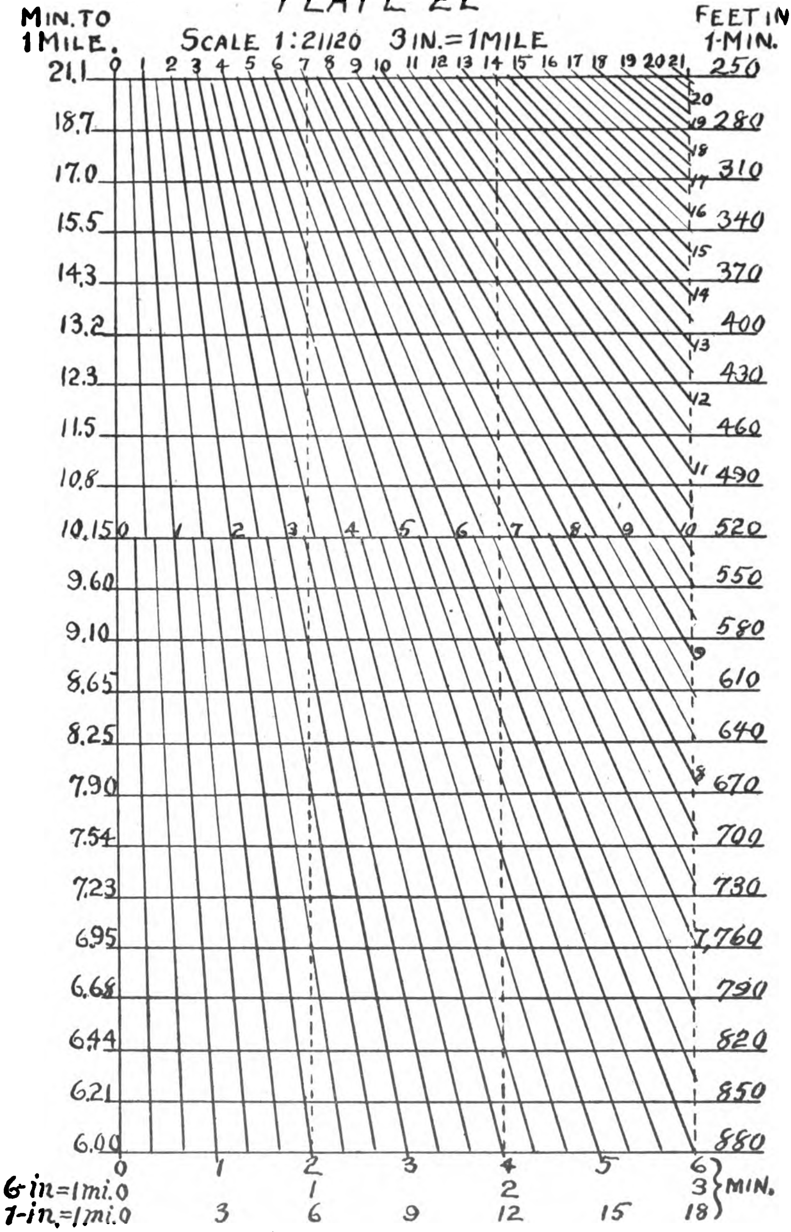

STEP OR STRIDE.
SCALE 1 : 21120 3-in.=1-mi.
49" 50" 51" 52" 53" 54" 55" 56" 57" 58" 59" 60" 61" 62" 63" 64" 65" 66" 67" 68" 69" 70" 71" 72" 73"
100 50 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 1500
1500 1450 1400 1400 1350 1350 1300 1300 1250 1250 1200 1200 1200 1150 1150 1100 1100 1100 1050 1050 1050 1050 1000
100 50 0 1 2 3 4 5 6 7 8 9 1000
6-in.=1 mi. 0 1 2 3 4 500
1-in=1 mi. 0 3 6 9 12 15 18 21 24 27 3000.
PACES

PLATE 21.

PLATE 22


MIN. TO 1 MILE.
SCALE 1:21120 3 IN.=1 MILE
FEET IN 1-MIN.
21.1
18.7
17.0
15.5
14.3
13.2
12.3
11.5
10.8
10.15
9.60
9.10
8.65
8.25
7.90
7.54
7.23
6.95
6.68
6.44
6.21
6.00
250
280
310
340
370
400
430
460
490
520
550
580
610
640
670
700
730
760
790
820
850
880
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21
0 1 2 3 4 5 6 7 8 9 10
0 1 2 3 4 5 6
6 in=1 mi. 0 1 2 3
1-in.=1 mi. 0 3 6 9 12 15 18
MIN.

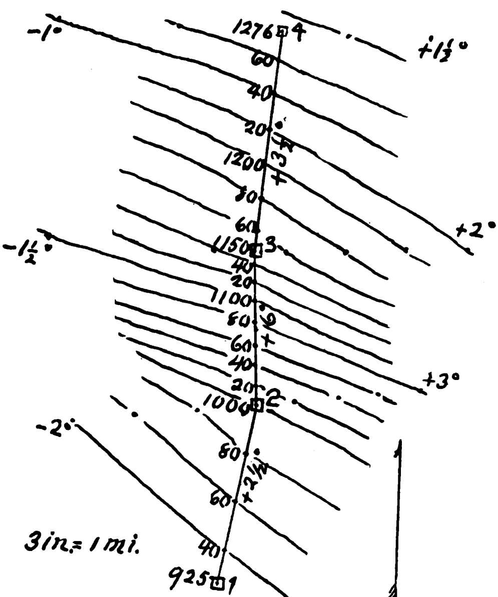

1276 4
60
40
20
1200
80
60
1150 3
40
20
1100
80
60
40
20
1000 2
80
60
40
925 1
-1°
+1½°
+3½°
+2°
-1½°
+1½°
+3°
-2°
+2½°
3in.= 1mi.

PLATE 23.

TABLE.

Horizontal Distances and Elevations From Stadia Readings.

Minutes.	0°		1°		2°		3°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	100.00	0.00	99.97	1.74	99.88	3.49	99.73	5.23
2	100.00	0.06	99.97	1.80	99.87	3.55	99.72	5.28
4	100.00	0.12	99.97	1.86	99.87	3.60	99.71	5.34
6	100.00	0.17	99.96	1.92	99.87	3.66	99.71	5.40
8	100.00	0.23	99.96	1.98	99.86	3.72	99.70	5.46
10	100.00	0.29	99.96	2.04	99.86	3.78	99.69	5.52
12	100.00	0.35	99.96	2.09	99.85	3.84	99.69	5.57
14	100.00	0.41	99.95	2.15	99.85	3.90	99.68	5.63
16	100.00	0.47	99.95	2.21	99.84	3.95	99.68	5.69
18	100.00	0.52	99.95	2.27	99.84	4.01	99.67	5.75
20	100.00	0.58	99.95	2.33	99.83	4.07	99.66	5.80
22	100.00	0.64	99.94	2.38	99.83	4.13	99.66	5.86
24	100.00	0.70	99.94	2.44	99.82	4.18	99.65	5.92
26	99.99	0.76	99.94	2.50	99.82	4.24	99.64	5.98
28	99.99	0.81	99.93	2.56	99.81	4.30	99.63	6.04
30	99.99	0.87	99.93	2.62	99.81	4.36	99.63	6.09
32	99.99	0.93	99.93	2.67	99.80	4.42	99.62	6.15
34	99.99	0.99	99.93	2.73	99.80	4.48	99.62	6.21
36	99.99	1.05	99.92	2.79	99.79	4.53	99.61	6.27
38	99.99	1.11	99.92	2.85	97.79	4.59	99.60	6.33
40	99.99	1.16	99.92	2.91	99.78	4.65	99.59	6.38
42	99.99	1.22	99.91	2.97	99.78	4.71	99.59	6.44
44	99.98	1.28	99.91	3.02	99.77	4.76	99.58	6.50
46	99.98	1.34	99.90	3.08	99.77	4.82	99.57	6.56
48	99.98	1.40	99.90	3.14	99.76	4.88	99.56	6.61
50	99.98	1.45	99.90	3.20	99.76	4.94	99.56	6.67
52	99.98	1.51	99.89	3.26	99.75	4.99	99.55	6.73
54	99.98	1.57	99.89	3.31	99.74	5.05	99.54	6.78
56	99.97	1.63	99.89	3.37	99.74	5.11	99.53	6.84
58	99.97	1.69	99.88	3.43	99.73	5.17	99.52	6.90
60	99.97	1.74	99.88	3.49	99.73	5.23	99.51	6.96
$c=0.75$	0.75	0.01	0.75	0.02	0.75	0.03	0.75	0.05
$c=1.00$	1.00	0.01	1.00	0.03	1.00	0.04	1.00	0.06
$c=1.25$ ...	1 25	0.02	1.25	0.03	1.25	0.05	1.25	0.08

TABLE — *Continued.*

HORIZONTAL DISTANCES AND ELEVATIONS FROM STADIA READINGS.

Minutes.	4°		5°		6°		7°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	99.51	6.96	99.24	8.68	98.91	10.40	98.51	12.10
2	99.51	7.02	99.23	8.74	98.90	10.45	98.50	12.15
4	99.50	7.07	99.22	8.80	98.88	10.51	98.48	12.21
6	99.49	7.13	99.21	8.85	98.87	10.57	98.47	12.26
8	99.48	7.19	99.20	8.91	98.86	10.62	98.46	12.32
10	99.47	7.25	99.19	8.97	98.85	10.68	98.44	12.38
12	99.46	7.30	99.18	9.03	98.83	10.74	98.43	12.43
14	99.46	7.36	99.17	9.08	98.82	10.79	98.41	12.49
16	99.45	7.42	99.16	9.14	98.81	10 85	98.40	12.55
18	99.44	7.48	99.15	9.20	98.80	10.91	98.39	12.60
20	99.43	7.53	99.14	9.25	98.78	10.96	98.37	12.66
22	99.42	7.59	99.13	9.31	98.77	11.02	98.36	12.72
24	99.41	7.65	99.11	9.37	98.76	11.08	98.34	12.77
26	99.40	7.71	99.10	9.43	98.74	11.13	98.33	12.83
28	99.39	7.76	99.09	9.48	98.73	11.19	98.31	12.88
30	99.38	7.82	99.08	9.54	98.72	11.25	98.29	12.94
32	99.38	7.88	99.07	9.60	98.71	11.30	98.28	13.00
34	99.37	7.94	99.06	9.65	98.69	11.36	98.27	13.05
36	99.36	7.99	99.05	9.71	98.68	11.42	98.25	13.11
38	99.35	8.05	99.04	9.77	98.67	11.47	98.24	13.17
40	99.34	8.11	99.03	9.83	98.65	11.53	98.22	13.22
42	99.33	8.17	99.01	9.88	98.64	11.59	98.20	13.28
44	99.32	8.22	99.00	9.94	98.63	11.64	98.19	13.33
46	99.31	8.28	98.99	10.00	98.61	11.70	98.17	13.39
48	99.30	8.34	98.98	10.05	98.60	11.76	98.16	13.45
50	99.29	8.40	98.97	10.11	98.58	11.81	98.14	13.50
52	99.28	8.45	98.96	10.17	98.57	11.87	98.13	13.56
54	99.27	8.51	98.84	10.22	98.56	11.93	98.11	13.61
56	99.26	8.57	98.93	10.28	98.54	11.98	98.10	13.67
58	99.25	8.63	98.92	10.34	98.53	12.04	98.08	13.73
60	99.24	8.68	98.91	10.40	98.51	12.10	98.06	13.78
$c = 0.75$......	0.75	0.06	0.75	0.07	0.75	0.08	0.74	0.10
$c = 1.00$......	1.00	0.08	0.99	0.09	0.99	0.11	0.99	0.13
$c = 1.25$......	1.25	0.10	1.24	0.11	1.24	0.14	1.24	0.16

TABLE — *Continued.*

Horizontal Distances and Elevations From Stadia Readings.

Minutes.	8°		9°		10°		11°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist	Diff. Elev.
0	98.06	13.78	97.55	15.45	96.98	17.10	96.36	18.73
2	98.05	13.84	97.53	15.51	96.96	17.16	96.34	18.78
4	98.03	13.89	97.52	15.56	96.94	17.21	96.32	18.84
6	98.01	13.95	97.50	15.62	96.92	17.26	96.29	18.89
8	98.00	14.01	97.48	15.67	96.90	17.32	96.27	18.95
10	97.98	14.06	97.46	15.73	96.88	17.37	96.25	19.00
12	97.97	14.12	97.44	15.78	96.86	17.43	96.23	19.05
14	97.95	14.17	97.43	15.84	96.84	17.48	96.21	19.11
16	97.93	14.23	97.41	15.89	96.82	17.54	96.18	19.16
18	97.92	14.28	97.39	15.95	96.80	17.59	96.16	19.21
20	97.90	14.34	97.37	16.00	96.78	17.65	96.14	19.27
22	97.88	14.40	97.35	16.06	96.76	17.70	96.12	19.32
24	97.87	14.45	97.33	16.11	96.74	17.76	96.09	19.38
26	97.85	14.51	97.31	16.17	96.72	17.81	96.07	19.43
28	97.83	14.56	97.29	16.23	96.70	17.86	96.05	19.48
30	97.82	14.62	97.28	16.28	96.68	17.92	96.03	19.54
32	97.80	14.67	97.26	16.33	96.66	17.97	96.00	19.59
34	97.78	14.73	97.24	16.39	96.64	18.03	95.98	19.64
36	97.76	14.79	97.22	16.44	96.62	18.08	95.96	19.70
38	97.75	14.84	97.20	16.50	96.60	18.14	95.93	19.75
40	97.73	14.90	97.18	16.55	96.57	18.19	95.91	19.80
42	97.71	14.95	97.16	16.61	96.55	18.24	95.89	19.86
44	97.69	15.01	97.14	16.66	96.53	18.30	95.86	19.91
46	97.68	15.06	97.12	16.72	96.51	18.35	95.84	19.96
48	97.66	15.12	97.10	16.77	96.49	18.41	95.82	20.02
50	97.64	15.17	97.08	16.83	96.47	18.46	95.79	20.07
52	97.62	15.23	97.06	16.88	96.45	18.51	95.77	20.12
54	97.61	15.28	97.04	16.94	96.42	18.57	95.75	20.18
56	97.59	15.34	97.02	16.99	96.40	18.62	95.72	20.23
58	97.57	15.40	97.00	17.05	96.38	18.68	95.70	20.28
60	97.55	15.45	96.98	17.10	96.36	18.73	95.68	20.34
$c = 0.75$......	0.74	0.11	0.74	0.12	0.74	0.14	0.73	0.15
$c = 1.00$......	0.99	0.15	0.99	0.16	0.98	0.18	0.98	0 20
$c = 1.25$......	1.23	0.18	1.23	0.21	1.23	0.23	1.22	0.25

TABLE—*Continued.*

HORIZONTAL DISTANCES AND ELEVATIONS FROM STADIA READINGS.

Minutes.	12°		13°		14°		15°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	95.68	20.34	94.94	21.92	94.15	23.47	93.30	25.00
2	95.65	20.39	94.91	21.97	94.12	23.52	93.27	25.05
4	95.63	20.44	94.89	22.02	94.09	23.58	93.24	25.10
6	95.61	20.50	94.86	22.08	94.07	22.63	93.21	25.15
8	95.58	20.55	94.84	22.13	94.04	23.68	93.18	25.20
10	95.56	20.60	94.81	22.18	94.01	23.73	93.16	25.25
12	95.53	20.66	94.79	22.23	93.98	23.78	93.13	25.30
14	95.51	20.71	94.76	22.28	93.95	23.83	93.10	25.35
16	95.49	20.76	94.73	22.34	93.93	23.88	93.07	25.40
18	95.46	20.81	94.71	22.39	93.90	23.93	93.04	25.45
20	95.44	20.87	94.68	22.44	93.87	23.99	93.01	25.50
22	95.41	20.92	94.66	22.49	93.84	24.04	92.98	25.55
24	95.39	20.97	94.63	22.54	93.81	24.09	92.95	25.60
26	95.36	21.03	94.60	22.60	93.79	24.14	92.92	25.65
28	95.34	21.08	94.58	22.65	93.76	24.19	92.89	25.70
30	95.32	21.13	94.55	22.70	93.73	24.24	92.86	25.75
32	95.29	21.18	94.52	22.75	93.70	24.29	92.83	25.80
34	95.27	21.24	94.50	22.80	93.67	24.34	92.80	25.85
36	95.24	21.29	94.47	22.85	93.65	24.39	92.77	25.90
38	95.22	21.34	94.44	22.91	99.62	24.44	92.74	25.95
40	95.19	21.39	94.42	22.96	93.59	24.49	92.71	26.00
42	95.17	21.45	94.39	23.01	93.56	24.55	92.68	26.05
44	95.14	21.50	94.36	23.06	93.53	24.60	92.65	26.10
46	95.12	21.55	94.34	23.11	93.50	24.65	92.62	26.15
48	95.09	21.60	94.31	23.16	93.47	24.70	92.59	26.20
50	95.07	21.66	94.28	23.22	93.45	24.75	92.56	26.25
52	95.04	21.71	94.26	23.27	93.42	24.80	92.53	26.30
54	95.02	21.76	94.23	23.32	93.39	24.85	92.49	26.35
56	94.99	21.81	94.20	23.37	93.36	24.90	92.46	26.40
58	94.97	21.87	94.17	23.42	93.33	24.95	92.43	26.45
60	94.94	21.92	94.15	23.47	93.30	25.00	92.40	26.50
$c = 0.75$	0.73	0.16	0.73	0.17	0.73	0.19	0.72	0.20
$c = 1.00$	0.98	0.22	0.97	0.23	0.97	0.25	0.96	0.27
$c = 1.25$	1.22	0.27	1.21	0.29	1.21	0.31	1.20	0.34

TABLE — *Continued.*

HORIZONTAL DISTANCES AND ELEVATIONS FROM STADIA READINGS.

MINUTES.	16°		17°		18°		19°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	92.40	26.50	91.45	27.96	90.45	29.39	89.40	30.78
2	92.37	26.55	91.42	28.01	90.42	29.44	89.36	30.83
4	92.34	26.59	91.39	28.06	90.38	29.48	89.33	30.87
6	92.31	26.64	91.35	28.10	90.35	29.53	89.29	30.92
8	92.28	26.69	91.32	28.15	90.31	29.58	89.26	30.97
10	92.25	26.74	91.29	28.20	90.28	29.62	89.22	31.01
12	92.22	26.79	91.26	28.25	90.24	29.67	89.18	31.06
14	92.19	26.84	91.22	28.30	90.21	29.72	89.15	31.10
16	92.15	26.89	91.19	28.34	90.18	29.76	89.11	31.15
18	92.12	26.94	91.16	28.39	90.14	29.81	89.08	31.19
20	92.09	26.99	91.12	28.44	90.11	29.86	89.04	31.24
22	92.06	27.04	91.09	28.49	90.07	29.90	89.00	31.28
24	92.03	27.09	91.06	28.54	90.04	29.95	88.96	31.33
26	92.00	27.13	91.02	28.58	90.00	30.00	88.93	31.38
28	91.97	27.18	90.99	28.63	89.97	30.04	88.89	31.42
30	91.93	27.23	90.96	28.68	89.93	30.09	88.86	31.47
32	91.90	27.28	90.92	28.73	89 90	30.14	88.82	31.51
34	91.87	27.33	90.89	28.77	89.86	30.19	88.78	31.56
36	91.84	27.38	90.86	28.82	89.83	30.23	88.75	31.60
38	91.81	27.43	90.82	28.87	89.79	30.28	88.71	31.65
40	91.77	27.48	90.79	28.92	89.76	30.32	88.67	31.69
42	91.74	27.52	90.76	28.96	89.72	30.37	88.64	31.74
44	91.71	27.57	90.72	29.01	89.69	30.41	88.60	31.78
46	91.68	27.62	90.69	29.06	89.65	30.46	88.56	31.83
48	91.65	27.67	90.66	29.11	89.61	30.51	88.53	31.87
50	91.61	27.72	90.62	29.15	89.58	30.55	88.49	31.92
52	91.58	27.77	90.59	29.20	89.54	30.60	88.45	31.96
54	91.55	27.81	90.55	29.25	89.51	30.65	87.41	32.01
56	91.52	27.86	90.52	29.30	89.47	30.69	88.38	32.05
58	91.48	27.91	90.48	29.34	89.44	30.74	88.34	32.09
60	91.45	27.96	90.45	29.39	89.40	30.78	88.30	32.14
$c = 0.75$.....	0.72	0.21	0.72	0.23	0.71	0.24	0.71	0.25
$c = 1.00$......	0.86	0.28	0.95	0.30	0.95	0.32	0.94	0.33
$c = 1.25$....	1.20	0.35	1.19	0.38	1.19	0.40	1.19	0.42

TABLE—*Continued.*

HORIZONTAL DISTANCES AND ELEVATIONS FROM STADIA READINGS.

Minutes.	20°		21°		22°		23°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	88.30	32.14	87.16	33.46	85.97	34.73	84.73	35.97
2	88.26	32.18	87.12	33.50	85.93	34.77	84.69	36.01
4	88.23	32.23	87.08	33.54	85.89	34.82	84.65	36.05
6	88.19	32.27	87.04	33.59	85.85	34.86	84.61	36.09
8	88.15	32.32	87.00	33.63	85.80	34.90	84.57	36.13
10	88.11	32.36	86.96	33.67	85.76	34.94	84.52	36.17
12	88.08	32.41	86.92	33.72	85.72	34.98	84.48	36.21
14	88.04	32.45	86.88	33.76	85.68	35.02	84.44	36.25
16	88.00	32.49	86.84	33.80	85.64	35.07	84.40	36.29
18	87.96	32.54	86.80	33.84	85.60	35.11	84.35	36.33
20	87.93	32.58	86.77	33.89	85.56	35.15	84.31	36.37
22	87.89	32.63	86.73	33.93	85.52	35.19	84.27	36.41
24	87.85	32.67	86.69	33.97	85.48	35.23	84.23	36.45
26	87.81	32.72	86.65	34.01	85.44	35.27	84.18	36.49
28	87.77	32.76	86.61	34.06	85.40	35.31	84.14	36.53
30	87.74	32.80	86.57	34.10	85.36	35.36	84.10	36.57
32	87.70	32.85	86.53	34.14	85.31	35.40	84.06	36.61
34	87.66	32.89	86.49	34.18	85.27	35.44	84.01	36.65
36	87.62	32.93	86.45	34.23	85.23	35.48	83.97	36.69
38	87.58	32.98	86.41	34.27	85.19	35.52	83.93	36.73
40	87.54	33.02	86.37	34.31	85.15	35.56	83.89	36.77
42	87.51	33.07	86.33	34.35	85.11	35.60	83.84	36.80
44	87.47	33.11	86.29	34.40	85.07	35.64	83.80	36.84
46	87.43	33.15	86.25	34.44	85.02	35.68	83.76	36.88
48	87.39	33.20	86.21	34.48	84.98	35.72	83.72	36.92
50	87.35	33.24	86.17	34.52	84.94	35.76	83.67	36.96
52	87.31	33.28	86.13	34.57	84.90	35.80	83.63	37.00
54	87.27	33.33	86.09	34.61	84.86	35.85	83.59	37.04
56	87.24	33.37	86.05	34.65	84.82	35.89	83.54	37.08
58	87.20	33.41	86.01	34.69	84.77	35.93	83.50	37.12
60	87.16	33.46	85.97	34.73	84.73	35.97	83.46	37.16
$c = 0.75$	0.70	0.26	0.70	0.27	0.69	0.29	0.69	0.30
$c = 1.00$	0.94	0.35	0.93	0.37	0.92	0.38	0.92	0.40
$c = 1.25$	1.17	0.44	1.16	0.46	1.15	0.48	1.15	0.50

TABLE — *Continued.*

Horizontal Distances and Elevations From Stadia Readings.

Minutes.	24°		25°		26°		27°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	83.46	37.16	82.14	38.30	80.78	39.40	79.39	40.45
2	83.41	37.20	82.09	38.34	80.74	39.44	79.34	40.49
4	83.37	37.23	82.05	38.38	80.69	39.47	79.30	40.52
6	83.33	37.27	82.01	38.41	80.65	39.51	79.25	40.55
8	83.28	37.31	81.96	38.45	80.60	39.54	79.20	40.59
10	83.24	37.35	81.92	38.49	80.55	39.58	79.15	40.62
12	83.20	37.39	81.87	38.53	80.51	39.61	79.11	40.66
14	83.15	37.43	81.83	38.56	80.46	39.65	79.06	40.69
16	83.11	37.47	81.78	38.60	80.41	39.69	79.01	40.72
18	83.07	37.51	81.74	38.64	80.37	39.72	78.96	40.76
20	83.02	37.54	81.69	38.67	80.32	39.76	78.92	40.79
22	82.98	37.58	81.65	38.71	80.28	39.79	78.87	40.82
24	82.93	37.62	81.60	38.75	80.23	39.83	78.82	40.86
26	82.89	37.66	81.56	38.78	80.18	39.86	78.77	40.89
28	82.85	37.70	81.51	38.62	80.14	39.90	78.73	40.92
30	82.80	37.74	81.47	38.86	80.09	39.93	78.68	40.96
32	82.76	37.77	81.42	38.89	80.04	39.97	78.63	40.99
34	82.72	37.81	81.38	38.93	80.00	40.00	78.58	41.02
36	82.67	37.85	81.33	38.97	79.95	40.04	78.54	41.06
38	82.63	37.89	81.28	39.00	79.90	40.07	78.49	41.09
40	82.58	37.93	81.24	39.04	79.86	40.11	78.44	41.12
42	82.54	37.96	81.19	39.08	79.81	40.14	78.39	41.16
44	82.49	38.00	81.15	39.11	79.76	40.18	78.34	41.19
46	82.45	38.04	81.10	39.15	79.72	40.21	78.30	41.22
48	82.41	38.08	81.06	39.18	79.67	40.24	78.25	41.26
50	82.36	38.11	81.01	39.22	79.62	40.28	78.20	41.29
52	82.32	38.15	80.97	39.26	79.58	40.31	78.15	41.32
54	82.27	38.19	80.92	39.29	79.53	40.35	78.10	41.35
56	82.23	38.23	80.87	39.33	79.48	40.38	78.06	41.39
58	82.18	38.26	80.83	39.36	79.44	40.42	78.01	41.42
60	82.14	38.30	80.78	39.40	79.39	40.45	77.96	41.45
$c = 0.75$	0.68	0.31	0.68	0.32	0.67	0.33	0.66	0.35
$c = 1.00$	0.91	0.41	0.90	0.43	0.89	0.45	0.89	0.46
$c = 1.25$	1.14	0.52	1.13	0.54	1.12	0.56	1.11	0.58

TABLE — *Continued.*

HORIZONTAL DISTANCES AND ELEVATIONS FROM STADIA READINGS.

Minutes.	28°		29°		30°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	77.96	41.45	76.50	42.40	75.00	43.30
2	77.91	41.48	76.45	42.43	74.95	43.33
4	77.86	41.52	76.40	42.46	74.90	43.36
6	77.81	41.55	76.35	42.49	74.85	43.39
8	77.77	41.58	76.30	42.53	74.80	43.42
10	77.72	41.61	76.25	42.56	74.75	43.45
12	77.67	41.65	76.20	42.59	74.70	43.47
14	77.62	41.68	76.15	42.62	74.65	43.50
16	77.57	41.71	76.10	42.65	74.60	43.53
18	77.52	41.74	76.05	42.68	74.55	43.56
20	77.48	41.77	76.00	42.71	74.49	43.59
22	77.42	41.81	75.95	42.74	74.44	43.62
24	77.38	41.84	75.90	42.77	74.39	43.65
26	77.33	41.87	75.85	42.80	74.34	43.67
28	77.28	41.90	75.80	42.83	74.29	43.70
30	77.23	41.93	75.75	42.86	74.24	43.73
32	77.18	41.97	75.70	42.89	74.19	43.76
34	77.13	42.00	75.65	42.92	74.14	43.79
36	77.09	42.03	75.60	42.95	74.09	43.82
38	77.04	42.06	75.55	42.98	74.04	43.84
40	76.99	42.09	75.50	43.01	73.99	43.87
42	76.94	42.12	75.45	43.04	73.93	43.90
44	76.89	42.15	75.40	43.07	73.88	43.93
46	76.84	42.19	75.35	43.10	73.83	43.95
48	76.79	42.22	75.30	43.13	73.78	43.98
50	76.74	42.25	75.25	43.16	73.73	44.01
52	76.69	42.28	75.20	43.18	73.68	44.04
54	76.64	42.31	75.15	43.21	73.63	44.07
56	76.59	42.34	75.10	43.24	73.58	44.09
58	76.55	42.37	75.05	43.27	73.52	44.12
60	76.50	42.40	75.00	43.30	73.47	44.15
$c = 0.75$	0.66	0.36	0.65	0.37	0.65	0.38
$c = 1.00$	0.88	0.48	0.87	0.49	0.86	0.51
$c = 1.25$	1.10	0.60	1.09	0.62	1.08	0.64