The triangulation method of stadia transit topographic surveying adapted to landscape architecture

The Triangulation Method of Stadia Transit Topographic Surveying Adapted to Landscape Architecture Illustrated With A Practical Problem

by

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ILLUSTRATED WITH A PRACTICAL PROBLEM

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CINCINNATI, OHIO.

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THE TRIANGULATION METHOD OF STADIA TRANSIT TOPOGRAPHIC SURVEYING ADAPTED TO LANDSCAPE ARCHITECTURE

Topographic surveying as particularly applied to landscape architecture is a subject on which there is a noticeable meagreness of published information. Many books on plane surveying are available for the use of the landscape architect and such publications include complete discussions of topographic surveying. However these books are written primarily for the use of the civil engineer and deal with the subject from his point of view.

The civil engineer is more interested in the development of artificial features, such as bridges, waterworks, railroads, and the like, than in the development of natural features. Generally his problem is one of mass quantities of earthwork, with little or no attention being paid to the small variations in the terrain which are interesting to the landscape architect. Over a very large area these variations usually balance one another and therefore do not alter the engineer's estimate of earthwork quantities.

The basic principles and methods as used by the civil engineer are also used by the landscape architect, but specific instances of their application to his particular requirements and problems are not found in the literature on the subject. For this reason the landscape architect has gradually evolved numerous methods of variations of methods, and specific applications of principles, which are especially suited to his requirements.

It is the purpose of this paper to attempt to bridge the gap between the abundance of information on the subject of topographic surveying as a whole and the scarcity of information available to the landscape architect in his particular field. More or less unguided by text-books on this subject, the landscape architect has created certain procedures all his own which enable him, efficiently and economically, to make topographic surveys.

While these methods of surveying, as adapted by the landscape architect, are universally used, they vary in detail according to the conditions in different locations and in different landscape organizations.

It is safe to say that with any one method no two organizations use the same procedure as to detail. Therefore, no attempt will be made to describe all the variations used in the triangulation method of topographic surveying. The procedure described herein is sound and practicable, based upon the practice of one organization and not upon the whole field of landscape architecture. It may be followed explicitly or may be changed as to detail more nearly to suit conditions in other organizations.

An efficient topographic survey is one which fully serves every purpose for which it is made. Its value depends more upon the accuracy of the information which is represented rather than upon the minuteness or quantity of detail.

It is the purpose of the landscape architect to improve land for human use and enjoyment in such a way as to combine the maximum of utility with the maximum of beauty. Therefore all existing information which will assist him in accomplishing this aim should be shown by the topographic survey. Such information will include locations, descriptions and sizes of trees, location of swales, brooks, ledges and other related features; soil conditions, (marsh, gravel, etc.), character of undergrowth, mass foliage outlines where the growth is not of sufficient size or importance to warrant detailed location, possible house sites, photo stations and important views, and all artificial features such as fences, buildings and roads. Accurate elevations should also be shown for all of these features.

FIELD WORK

A well organized survey party should consist of one transitman, one note keeper and two rodmen, the chief of the party being any one of the four but preferably the note keeper. Every man in the party should be capable and experienced; that is of vital importance and determines the accuracy of the survey since one careless or incapable person practically nullifies the efforts of all the others. The chances of error in the field work of a survey are so numerous that each person is required to be responsible only for his own duties and cannot be expected to check the work of others. On any observation the transitman is liable to make an error through incorrect azimuth, distance or elevation, the notekeeper might erroneously record the azimuth, distance, elevation, description or some reference, and, finally, the rodman might hold the rod incorrectly, miscall trees, etc. or give incorrect references especially by confusing "right" and "left". These possible chances of error will be described in more detail on page 37.

Any standard make of transit with plate and telescope bubbles, stadia hairs as well as vertical and horizontal cross-hairs, vertical arc and with a six inch or seven inch circle graduated continuously from zero to three hundred sixty degrees is satisfactory for stadia transit topographic surveying. The telescope should be of excellent quality with flat field and good illumination; the magnifying power should be twenty-five or thirty. It is also advisable to use a collapsible tripod for ease in shipping as well as in regulating the height of the "set-up" in the field. A twelve foot, three section, collapsible Chicago rod, readable to hundredths of a foot, has also been found to be very satisfactory. A collapsible rod has the advantage of being easily handled in traveling and in the field where undergrowth will not permit the use of a long rod. One or more sections of the rod may be removed from the top for convenience in this kind of work.

For referencing elevations each rodman should be equipped with an ordinary hand or Locke level. All other material is the same as used by any well organized survey party.

The precision with which the angles and distances are read will be governed almost entirely by the scale of the map to be plotted. Intra-station azimuths should be read as closely as the vernier permits, since they are to be plotted by co-ordinates. On "side-shots" (readings other than to stations) the azimuth need not be read closer than about 5 minutes, because this is the limit of precision of the ordinary protractor used for plotting details. If the scale of the map is such that a distance of 1 foot can be plotted, then the stadia distances must be read as closely as possible. The corresponding azimuths should be read with a precision which depends upon the distance. For example, if the rod is 57 feet away and the azimuth is read only to the nearest whole degree, then the point is located within one foot both in distance and in azimuth; at a distance of 340 feet the nearest 10 minutes of azimuth corresponds to the nearest foot of distance; for 680 feet away, 5 minutes of azimuth corresponds to one foot.

It will usually be necessary to take the vertical angle to the nearest minute in order to obtain the elevations with 0.1 foot, which is the accuracy usually expected in this method. In a 340 foot sight an error of 1 minute of angle produces an error of 0.1 foot in elevation, hence on all except rough work the vertical angles should be read to the nearest minute. The elevations of important points should be determined from at least two different stations.

HORIZONTAL CONTROL

Before the actual work of surveying is begun the entire party should become familiar with the existing conditions to be encountered. The task of reconnoitering and planning the method to be followed is one which requires skill and experience, and the accuracy obtained in the final result depends largely upon the judgment used in laying out the stations.

In planning the field work of a topographic survey it should be remembered that while stadia measurements are sufficiently accurate for side shots or short traverses where the errors are not cumulative, nevertheless on long traverses these may become excessive. It is therefore desirable to establish the stations with a greater degree of accuracy in order to control the accuracy of the survey as a whole. This control may be obtained by any one of several methods.

The taped traverse, the stadia traverse and the triangulation are the three methods usually used. It is often advisable to use a combination of the three, especially over large ares of a diversified character. The method of taped traverse control is most commonly used on large wooded areas where detailed topography and tree location is desired, where the character of the vegetation or other natural features prohibit the use of the triangulation method, and where the method of stadia traverse is not sufficiently accurate. With this method, every intra-station line must be taped, thereby increasing the cost of the survey. More often one of the other two methods, usually stadia traverse, is used in connection with taped traverses. On large areas taped traverse lines may be carried around the boundary and triangulation or stadia traverse used in the center area not otherwise covered.

The method of stadia traverse is very seldom used alone. With this method all intra-station lines are measured by stadia and are not sufficiently accurate for detailed or important work. As mentioned before this method is most commonly used to fill in the less important details between taped traverse lines.

Triangulation is an application of the principles of trigonometry to the measurement of inaccessible lines and angles. As stated before, with the taped traverse method all the intra-station lines must be measured. This process is therefore more expensive than that of triangulation and usually is not likely to be so accurate. In a triangulation system one line, the base line, is measured directly; all the other distance are derived by measuring the angles of the triangles and calculating the sides by trigonometry.

To afford a check, however, two lines instead of one are usually measured. The intervening ground does not have to be traversed, so that the accuracy with which a distant station may be located is nearly independent of the character of the intervening country. In addition, where intra-station elevations are carried either trigonometrically or directly, the triangulation method affords a more complete network of levels, insuring a greater degree of accuracy in regard to vertical control. In ordinary survey work where the triangulation method is used the angles of each triangle are measured more than once, usually three or six times. However, when continuous azimuth is carried in connection with the triangulation method, each angle is necessarily measured only once. Therefore these measurements should be made with precision.

In measuring the base lines a steel tape or chain should be used. Great care should be exercised in holding the tape horizontal, in the plumbing, in the aligning, and in securing the proper tension. The tension should be kept uniform and preferably given with a spring balance. The temperature correction should be determined with an accuracy consistent with the requirements of the survey in question. There should be at least two measurements of each base line so as to afford a check, since an error in either base will affect the entire triangulation. On sloping ground inclined distances are measured as explained on page 33.

In the triangulation system of surveying equilateral triangles are ideal, but it is not probable that all triangles will be well shaped. If possible, however, none of the interior angles should be less than 45° or greater than 90°. Plate 5 shows the triangulation system of a survey. Triangles CDF and EHJ are examples of desirable proportions while EIJ and IJK are poorly proportioned. Whenever conditions permit all of the triangles in the network should be nearly uniform in the length of sides. However, the character of the terrain practically determines the sizes of triangles, as shown on Plate 7. The east end of the property is steeper and therefore the triangles are necessarily smaller.

Regardless of the method of horizontal control used it is advisable to carry continuous azimuth, thus affording frequent checks on the transit work thoroughout the entire field operations. By carrying continuous azimuth is meant a system of foresights and backsights such that like azimuths from all stations are always parallel and in the same direction. With continuous azimuth north is usually assumed as 0° and is 0° throughout the entire survey.

As illustrated on Plate 2 continuous azimuth is carried in the following manner: At Station "A" Magnetic North is assumed as 0°. The azimuth reading to station B is 25°-20' and to station D, 96°-13'. Then at station B, backsight is taken on station A with the plates set at 205°-20', or 25°-20' (azimuth from A to B) plus 180°-00'. At B the azimuth to station D is 131°-49'. Now at station D, backsight is taken on a previous station, either station A or B, but preferably on station A which is in this case the first station.

If the backsight is taken on station A, the plates will be set at 276°-13', which is 96°-13' (the azimuth from A to D), plus 180°-00'. With backsight on station B the plates will be set at 311°-49', the azimuth from B to D, (131°-49') plus 180°-00'. It will be noted that in each case 180°-00' has been added to the azimuth reading. This is true because each azimuth reading has been less than 180°-00'. If, however, the azimuth reading from station A to B were 225°-20', instead of 25°-20', 180°-00' would have been subtracted, making the backsight from station B to station A 45°-20', instead of 205°-20'. From this it will be noted that 180°-00' is added when the azimuth is less than 180°-00', and subtracted when the azimuth is greater than 180°-00'.

The actual selection of station points mentioned in connection with triangulation requires more skill and experience than any other part of the field work. The proper selection of stations reduces to a minimum the time required for field work and practically determines the accuracy of the survey. Poorly selected stations mean more stations in order to cover the area in a satisfactory manner, a poor triangulation system, and finally incorrect stadia readings because of obstructions. Station points must be selected in relation to one another so as to form good triangles and also in relation to terrain and other features which might obstruct sights. The entire survey party should assist the chief with this work. When the initial station has been selected, one member of the party is assigned to stand at the location so that he can be seen from other possible station locations. Then as the second and third locations are selected the next two in the party are assigned the same respective duties as that of the first. Thus the triangle is completed and it is assured that the other two stations can be seen from the third. The chief of the party can now select the fourth station location and signal the man on the first station to move forward. This same procedure is followed in selecting all other stations to complete the triangulation system. Stations are marked by the usual surveyor's hub, set flush with the ground, with an engineer's tack in the center of the top to designate the exact point.

VERTICAL CONTROL

In the ordinary triangulation system of a topographic survey the intra-station lines should be as nearly uniform in length as existing conditions will allow and none of the lines should be greater than 1000 feet. At this distance the decrease in the degree of accuracy with which readings can be taken materially reduces the accuracy of the survey.

Due to inevitable human errors, certain readings will be incorrect. A good transitman reduces these errors to a minimum but can never eliminate them entirely. Therefore, no reading should be considered seriously in which the difference in foresight and reverse sight varies more than 0.01 foot for each 100 feet of distance. The gain in accuracy in attempting to correct such readings does not justify the increased cost and loss of time. Smaller differences tend to balance each other and it is also a well known fact that the accuracy in plotting is much less than in computing.

Wherever extensive leveling is to be done it is necessary to have a system of reference points called bench marks (B.M.) the relative heights of which are accurately known. These heights are usually referred to some definite horizontal plane, such, for instance, as mean sea-level, and the height of a point above this plane is called its elevation. If mean sea-level datum is not known a datum can be arbitrarily assumed. A datum is any base to which the elevation of every point of a series is referred.

Whenever possible it is best to use a Government bench mark as a basis for the vertical control. The elevation of such a bench mark is based upon United States Geodetic Survey data and is the most nearly universal datum that can be obtained. Often one of these bench marks is sufficiently near the area being surveyed to warrant the running of a line of levels from it and establishing several other bench marks at convenient places. A survey based upon this uniform datum is more valuable than one based upon an assumed datum. This is true because the elevations shown are actual rather than comparative, and because the survey can be tied to any other survey of adjoining territory based upon the same datum even though the two surveys were made independently.

Often there is no Government bench mark to which a tie can be made. In this case it is necessary to assume some elevation for a permanent object to be used as the bench mark. This assumed elevation is usually 100.00, 500.00, or 1000.00, but more often 500.00. In any case, however, the assumed elevation should be greater than the difference in elevation between the bench mark and the lowest point on the survey. If this is done there will not be any points on the survey with a "minus" elevation (less than zero).

Leveling consists in ascertaining the differences in elevation, and there are two kinds, direct leveling and trigonometric leveling.

Direct leveling is the simplest and safest method to use over bench marks as well as over stations. This method eliminates all chances of error in reading vertical arcs. In fact, levels should never be carried trigonometrically when conditions are such that they can be carried directly.

The procedure followed in carrying intra-station elevations by direct leveling is practically the same as that used by the civil engineer in direct leveling. This procedure is as follows: The backsight added to the known elevation of the bench mark gives the "height of the instrument" (H.I.); the foresight subtracted from the height of the instrument gives the elevation of the point over which the rod is held.

It may be noted here that the terms foresight and backsight do not refer to the "forward" and "backward" directions. A backsight is a reading taken on a point of known elevation for the purpose of obtaining the height of the instrument. A foresight is a reading taken on a new point to determine its elevation. For this reason backsights are frequently called "plus sights" (+ S.), and foresights are called "minus sights" (-S.).

To expedite the field work, however,

inasmuch as the transit must be set up over each station to secure data for the horizontal control and the stadia readings (side shots), the elevation of the line of sight must be known. This is obtained at each station over which the transit is set by measuring with either a six foot rule or with the rod, from the top of the hub to the center of the eye-piece when the telescope is level. To correct this measurement to the true vertical distance between the top of the hub and the telescope axis, when using the ordinary size of transit, 0.01 ft. is subtracted for measurements over 5 feet and 0.02 foot for measurements under five feet in the ordinary range of set-ups. This method violates the cardinal leveling tenet of balancing the length of foresights and backsights, but this objection is overcome by securing the difference in elevation from both ends of each intra-station line. These differences in elevation are balanced as described under "Computations", page 52.

Where the difference in elevation between stations is too great to permit direct leveling in one setup it is necessary to carry trigonometric levels. This is done by observing the vertical angles between stations and computing the differences in elevation trigonometrically. These angles are taken at the time the station is occupied for the purpose of taking the stadia readings. The elevations of certain points in the survey are established by direct leveling from a known bench mark. From the elevations of these points the elevations of the stations may be found by means of the differences in height derived from the vertical angles and the lengths of the triangle sides. When reading vertical arcs it is advisable to sight as near to ground as possible and on an even foot except where atmospheric disturbances by heat waves detract from the accuracy. The method of computing these elevations is fully described under "Computations" on pages 53 and 54.

STADIA SIDE SHOTS

The stadia method of locating points has long been recognized as being particularly adapted to topographical surveys both as regards accuracy and economy. Topographic maps are of such character and are generally plotted on such scales that points cannot as a rule be located on the plan with any greater accuracy than they are located in the field by the stadia method.

As previously stated, a topographic survey made for the use of a landscape architect should show locations, descriptions and elevations of all those features which may assist him in performing his duties in a satisfactory manner. Such features include buildings, fences, wells, traveled ways, bridges, culverts, drainage lines and other artificial features, trees, undergrowth, mass foliage outlines, important views, possible house sites, springs, lakes, creeks, swales, ledges, marshes, etc., and all other natural features which may be important.

In topographical surveying by the stadia method points are located by means of (1) the azimuth, (2) the distance, and (3) the angle of elevation or depression. These points taken for the purpose of locating details are commonly called "side shots". The accuracy with which these measurements is taken need not be so great as that of the triangulation measurements, because an error in these measurements will affect only a single point, whereas an error in a triangulation line will be carried through the rest of the triangulation system.

Plate 1, a typical page from the level

book, shows the way field notes are recorded. The azimuth reading is recorded in the first column on the left side of the page. However, the azimuth is the last entry for each reading. This illustration shows the reverse chronological order of recording the notes in the field but the correct chronological order of plotting the notes.

Azimuth to the side shots is recorded only to the nearest five minutes. This is found to be of sufficient accuracy for plotting at the more generally used scales, of 20, 40, or 60 feet to the inch. Continuous azimuth is used with the side shots and in the same manner as with the traverse control.

The starting point for stadia measurements is not the center of the instrument. It is a point as far in front of the transit as is the focal length. The focal length, assumed as 1 foot, varies with different transits, but with the same transit is constant and must be added to each stadia distance reading. This focal correction is applied in the field directly to the stadia reading. In addition to this correction there are usually two others to be applied in order to obtain the true distance. These corrections are the stadia-hair correction which applies to both direct and trigonometric levels, and the vertical arc correction which applies only to trigonometric level readings. These two corrections are applied in the office and will be discussed more completely in a later paragraph.

When the stadia hairs are correctly adjusted there is no stadia-hair correction. However, there is usually a slight error and it is always advisable to test the instrument in the field. This is done by reading stadia distances on lines of known lengths and figuring the error on a percentage basis.

Where there is no stadia-hair correction and where direct levels were taken, the focal correction plus the stadia distance gives the true distance. These are the only conditions under which the true distance is determined in the field. Under other conditions they are computed in the office.

The vertical readings taken with stadia are of two kinds, direct levels and vertical arcs (angles of depression or elevation). The direct level reading subtracted from the height of the instrument gives the elevation of the point. For example, 502.4 (H.I.) minus 2.7 (-S) equals 499.7 (elevation). It will be noticed here that the height of the instrument and the direct level reading are given only to the nearest tenth of a foot. The height of the instrument is recorded to the nearest hundredth of a foot and is actually 502.36. However, for ordinary side shots the nearest tenth of a foot is of sufficient accuracy for both the height of the instrument and the level reading.

The correction for angles of depression (minus vertical arcs) and elevation (plus vertical arcs) are figured in the office and will be discussed under "Computations", pages 52 and 54 . Care should be taken, however, in recording the point on the rod at which the angle was taken. In most cases it is found advisable to take these readings at a point on the rod which is equal to the height of the instrument above the station. When this is done the angular correction can be applied directly to the station elevation. This differs from the method of taking vertical readings at stations as discussed on page 26 ; the former method is more accurate but involves more computations. The common sources of error in reading levels are given on page 37 .

"Remarks" and "references" are recorded in the notebook, remarks denoting a complete description of a point being located, while references are the location of points by their relation to some other point or points of known location. The rodmen are responsible for all remarks and references which are recorded, in the level book, before any stadia readings are taken. Plate 1 shows the precedence in making such notes. A more complete discussion of referencing is given on pages 35 and 36.

Views from station points in the general locations of proposed house sites are of special importance. These views are recorded from the stations and not from the exact locations of the proposed house sites. Each view is located by two azimuth readings which define the limits, and is described by a short note stating the outstanding characteristics.

POINTS REQUIRING SPECIAL CARE

Because the intra-station lines in the traverse control are computed from the measured base lines and the interior angles of the triangles, it is extremely important to secure well proportioned triangles with very accurate azimuth readings. These triangles (the traverse control) form the basis for the entire survey and the accuracy of the final results depends largely upon the work of locating stations so that they form well balanced triangles and upon the assurance of correct azimuth readings to these stations. Therefore, the importance of extreme care in this phase of the field work cannot be over emphasized. The requirements of well proportioned triangles are stated on page 18.

Great care should be used in securing a steady support for the instrument at all times, but more especially when reading trigonometric levels. A steady support is absolutely necessary or the transit cannot be brought to a true level. The transitman should examine the instrument at frequent intervals to determine whether it is still in adjustment and truly level. This is especially important where trigonometric readings are being taken because in the computations this error is amplified.

The triangulation base lines (lines AD and IJ, Plate 5), must be accurately measured because, with the interior angles of the triangles, they form the basis from which the horizontal control of the entire traverse system is computed. Whenever possible, the base lines should be selected so that they can be measured on a direct level. However, as with lines AD and IJ, this is not always the case. Line AD was measured on a direct level, while IJ, because of the great difference in elevation between the stations could not be so measured. In the latter case the "slope distance" method was used.

The method of slope distance measuring is usually employed where the difference in elevation between two stations is so great that the intra-station line cannot be measured on a direct level. With this method the measurement is generally taken from the axis of the telescope to the top of the station observed, and the vertical arc read to the same point. When that point cannot be seen it is necessary to measure to a point directly above the station to which the vertical arc was taken. Slope distance measuring is based upon the following formula: "The recorded distance multiplied by the cosine of the angle of depression (or elevation) equals the true horizontal distance."

Tapes are usually manufactured to be of standard length at 62° F. with a pull of 12 pounds on them while supported throughout their entire length. The average coefficient of expansion for a steel tape is nearly 0.0000063 for 1° F. Hence a change of temperature of 15° produces nearly 0.01 ft. change in the length of the tape. In ordinary survey work this correction is negligible and need not be considered very seriously. However, where long base lines are being measured in extremes of temperature this correction should be applied.

The method of measuring the height of the instrument above the station, together with its weakness and compensating features, was fully described on pages 24 and 25. Because of the apparent weakness, and because an error in reading or recording this measurement will effect every level reading from the station, it is doubly important that this measurement be correct.

At each station in the traverse control and immediately after the orienting station sight, an azimuth reading, or permanent backsight, should be taken on some outstanding permanent object, such as a flagpole or church spire. This backsight may be particularly useful in re-establishing the azimuth if the transit is moved or if the horizontal plates get out of adjustment after readings have been taken on other stations. When magnetic north is assumed as 0° azimuth and when the azimuth which is set on the horizontal plates is lost, it is practically impossible to re-establish 0° azimuth exactly as it was before. With a permanent backsight, however, whose azimuth is fixed and recorded as accurately as is practicable, the plates may be reset correctly without any difficulty and without having to duplicate any work such as readings to other stations.

By referencing is meant the location of points by their relations to some other point or points of known location. For example, a 4" Elm may be referenced from a 20" Maple being located as being 10 feet further, 2 feet right and 0.9 feet lower. It is preferable, however, actually to locate, by stadia, trees, roads, houses and such features, and from these points reference tops and bottoms of slopes, points in swales, ledges, etc. References shown on Plate 1 are typical. These same referenced points are shown plotted on Plate 6.

There are no set rules for referencing. Any method which is accurate and economical may be used. The ability to reference accurately and efficiently is based mainly upon experience. A point is referenced as being a certain number of feet nearer or further and to the right or left as viewed from the transit, and not to the right or left of the rodman. This orientation is used for its ease in plotting. To insure a greater degree of accuracy it has been found advisable to limit referenced distances to the right or left to a maximum of 10 feet, while the distance nearer or further may be longer. The former distances are measured at right angles to the line of sight and are liable to be grossly "off line" (right angles to the line of sight) when greater than 10 feet. The latter distances are measured on the line of sight and therefore all guesswork is eliminated. Measurements may be made with the regular rod without other equipment.

Vertical distances are much more difficult to reference than horizontal distances. It is advisable to use some kind of hand level or other similar instrument in order to be in keeping with the accuracy obtained in horizontal distance referencing. The common hand, or Locke, level is generally accepted for this use. The Locke level is a simple metal tube with plain glass covers at the ends and with a spirit level on top. When looking through the tube one sees the level bubble on one side of the tube in a mirror set at 45° with the line of sight, and the landscape on the other side. In order that the eye may see the bubble and the object at the same instant, the instrument is focused by means of a lens placed in a sliding tube. The level line is marked by a horizontal wire which can be adjusted by means of two screws. The instrument is held at the eye and the outer end is raised or lowered until the bubble is in the center of the tube. At this instant a point in line with the horizontal wire is noted, and in this way approximate levels are determined.

SOURCES OF ERRORS

The possible sources of errors in each side shot are so numerous that no attempt will be made to enumerate in detail every condition. A well organized party, all men of which are capable and experienced, reduces the errors to a minimum but can never eliminate them entirely. The main sources of errors are four, namely, incorrect azimuth, distance, elevation and description. It is sufficient to list the causes of these errors under their respective classifications.

An incorrect azimuth may be the result of an error from any of the following causes:

1. Changes due to temperature and wind
2. Uneven settling of tripod
3. Poor focusing
4. Inaccurate setting over point

5. Incorrect backsight

6. Reading the wrong direction on the circle

7. Reading azimuth 5° off

8. Reading azimuth 1° off, especially when nearly an even degree

9. Azimuth read as 116° instead of 16°

10. Reading of 310°-05' recorded as 31°-05'

An incorrect distance may be caused by an error in,

1. Not pulling chain or tape taut

2. Careless plumbing

3. Incorrect alignment

4. Effect of wind

5. Variation of temperature

6. Erroneous length of chain or tape

7. Transposing figures or reading tape upside down

8. Reading wrong foot mark

9. Reading one-half the intercept with stadia

10. Failure to add one foot for focal correction

Errors in elevation may be caused by,

1. Changes due to temperature and wind

2. Uneven settling of tripod

3. Reading the wrong cross-hair

4. Reading the wrong foot on the rod

5. Failure to read vertical arc

6. Reading or recording "plus" and "minus" vertical arcs interchanged

7. Reading vertical arc 1^{o} or 5^{o} off as with azimuth

8. Rod not held plumb

The common sources of errors in description and reference are,

1. Incorrect names and sizes of trees, etc.

2. Confusing directions and compass point

3. Incorrect distances in referencing

4. Interchanging "right" and "left" in referencing

5. Interchanging "higher" and "lower" in referencing

6. Giving reference or description with wrong reading

7. Notekeeper's errors in recording

CHECKING FIELD WORK

Regardless of the experience of the members of the survey party, and the care with which every detail of the field work was executed, some parts of the work will, naturally, be incorrect. Further checking of the field work, beyond the stage of a reasonable assurance that the work was executed correctly, must necessarily come after the readings have been plotted and the map practically completed, except for the inking and finishing. This is a part of the field work but comes, as stated before, after the office work. At least one member of the field party should check, or assist in checking, the survey. This method of checking the field work is primarily a method of inspection, in which the location of trees, buildings, ledges, swales, etc. as shown on the map are compared with the actual location. A Locke level, a box tape and a six-foot rule are the only instruments used in the field check. If the location of some feature as shown on the map is questionable it can be checked by two intersecting measurements taped from points which have been accepted as correctly located. Also, questionable elevations may be roughly checked with the Locke level and the six-foot rule, using as a base of levels any located point whose elevation is acceptable.

OFFICE WORK

The office party of the survey should consist of two men, preferably the notekeeper and transitman of the field party. These men will naturally be more familiar with the actual conditions of the area as well as the arrangement of the notes. They, therefore, can accomplish the office work more accurately and economically than others. The more experienced of the two will do most of the computing while the other will make the key sketch (page 61) and prepare the map for plotting. It is assumed that the transitman is the more experienced in computing. He will balance the angular closures, compute the intra-station lines and the co-ordinates, while the notekeeper prepares the key sketch to determine the size and "layout" of the map and plots the co-ordinate lines on this map. At this stage the notekeeper can plot the stations by co-ordinates, recently computed by the other member of the party, and begin the reduction of the stadia notes while the transitman figures the levels and balances the station elevations. After the transitman has finished these computations he will assist the notekeeper until they complete the stadia reductions, after which the actual plotting can be begun.

In most cases the survey is plotted in the headquarters office. This necessitates an additional trip to the area to check the field work but under ordinary conditions the convenience and accessibility to the requisite instruments and materials used in plotting more than offset this disadvantage. However, if the area is a very great distance from the headquarters office, the expense of an additional trip for one man to check the survey may prohibit this procedure. When this is the case it has been found satisfactory to plot the survey in the field office, that is, on the site.

Survey plans are made with greater accuracy than other plans of a landscape architect. It is evident then that additional instruments which may not be necessary in connection with other work are required in the computation and the plotting of a survey. The most common of these are as follows: the calculating machine, slide rule, straight-edge, engineer's scale, protractor and beam compass. The calculating machine is used in the computations of both the horizontal and vertical control of the survey. Any standard machine reading to six or eight places is satisfactory. The stadia slide rule is used in figuring side shots where vertical arcs are recorded. Their principles and application will be explained more completely under "Computations". The use of the ordinary instruments such as the steel straight edge, beam compass, engineer's scale and protractor is obvious. Special instances of their application will be explained under "Plotting and Finishing", page 60 .

COMPUTATIONS

HORIZONTAL CONTROL

Of the three methods of horizontal control mentioned on page 15 the choice for this particular problem is the triangulation method which will be discussed here.

In the triangulation network of the survey the sum of the interior angles in each triangle must equal 180°. When this is the case the triangle is said to be closed, or correct. However, errors are inevitable and the triangle is not always closed without adjustment. When the sum of the interior angles is not 180° it is necessary to "balance" the triangle by distributing among these angles the number of minutes or seconds that the total varies from 180°. This work is simplified by the use of a sketch (Plate 2). This sketch need not be drawn to a definite scale but should show, in a general way, the proportional sizes and chapes of triangles. On this sketch should be written, as shown, the azimuth readings as recorded in the field work. Then, from these azimuth readings, the interior angles of each triangle are figured by subtracting the lesser azimuth reading from the greater. For example: At station B, the azimuth reading to station C is 89°-05'; to D, 131°-49'; and to A, 205°-20'. Then 89°-05' subtracted from 131°-49' gives 42°-44', the interior angle CBD; and 131°-49', subtracted from 205°-20' gives 73°-31', the interior angle DBA. These interior angles should then be recorded around the station from which they were determined and between the intra-station lines to the other two stations which complete the triangle. After all of these interior angles have been determined and recorded, each triangle is balanced, as outlined in the following example: The interior angles of triangle ABD are 70°-53', 73°-31' and 35°-35'-30", for A, B and D respectively; and the interior angles of triangle BCD are 42°-44', 94°-23', and 42°-52' for B, C, and D respectively. Neither of these triangles is "closed", each having an error of -1'. It is then assumed that the error is caused by obscure sights from A to D and from B to C, leaving line BD, common to both triangles, unchanged. Therefore, the two triangles are balanced by changing the azimuth readings of lines BC and AD, which, naturally, also changes the interior angles BAD, ADB, (from line AD), CBD and BCD (from line BC). In this case, 30" was added to each angle to offset the error of -1 minute in each triangle, there being two such changed angles in each of the two triangles. These two triangles are now said to be balanced, and cannot be changed again to assist in balancing other interior angles. Line CD, being common to both triangle BCD and CDF has been determined and accepted as correct. Therefore any changes in triangle CDF will, naturally, have to be made in lines CF and DF. In balancing it will often be found that of two triangles having a common side the sum of the angles in one will be slightly more than 180°, while the sum of the angles in the other will be less than 180° by approximately the same amount. This indicates a possible error in reading the azimuth in the field between the two stations at the ends of the line, and balancing is easily accomplished by a deduction from one interior angle and the addition of a like amount to the interior angle on the other side of the common line. In any event the sum of the angles about any given station must be 360°. These corrected interior angles are used in computing the azimuths and bearings of the station lines, and the co-ordinates. In this same general manner the entire triangulation control is balanced. Table 1 shows in a concise form how this angular balance is accomplished.

The process of computing the corrected azimuths of all intra-station lines is the reverse of the calculation of interior angles. Accepting the azimuth of some line, usually the preliminary measured base line, to be correct as observed, the azimuths of other intra-station lines are secured by adding or subtracting the balanced interior angles, depending upon whether the deflections were to the right or to the left. In the present instance, (Plate 2), the azimuth of line BD was accepted as a correct observed azimuth so that the deviation of the balanced azimuths from the observed azimuths throughout the system might be as little as possible.

In the triangulation system one line, the base line, is measured directly. Having determined this length and the corrected interior angles, the lengths of all other intra-station lines are figured by a trigonometric formula known as the Law of Sines and stated $\frac{\text{Sin A}}{\text{Sin B}} = \frac{a}{b}$, or Side a (given) divided by the sine of the angle opposite side a, and multiplied by the sine of the angle opposite b (to be determined) equals side b. This formula is usually stated in the more usuable form,

$$\frac{a}{\sin A} \times \sin B = b,$$

$$\frac{a}{\sin A} \times \sin C = c,$$

Triangle ABD shown on Plate 3 supplies an illustration of the computation of the triangle sides. The corrected angles are

$$A = 70^{\circ}\text{-}53'\text{-}30''$$

$$B = 73^{\circ}\text{-}31'\text{-}00''$$

$$D = 35^{\circ}\text{-}35'\text{-}30''$$

Side AD, 655.65 ft. is given

$$\frac{655.65 \times \sin 35^{\circ}\text{-}35'\text{-}30''}{\sin 73^{\circ}\text{-}31'} = \text{side AB}$$

$$\frac{655.65 \times .582005}{.958902} = \text{side AB}$$

$$683.751 \times .582005 = 397.95 \text{ ft. (AB)}$$

Solving for side BD,

$$\frac{655.65 \times \sin 70^{\circ}\text{-}53'\text{-}30''}{\sin 73^{\circ}\text{-}31'} = \text{side BD}$$

$$\frac{655.65 \times .944901}{.958902} = \text{side BD}$$

$$683.751 \times .944901 = 646.08 \text{ ft. (BD)}$$

Side BD, having now been determined, can be used in figuring the other sides in the triangle BCD. Thus all sides in the triangulation system can be determined in the same manner.

It has been stated that only one line measured directly is needed to figure the other intra-station lines. As a general practice, however, it is recommended that two lines be measured primarily as a check on the accuracy of the computed lines. Those two measured lengths should be used to figure through to some common side. As shown on Plate 3, line FE is figured from line AD through BD, CD and FD; and from IJ through HI, GI and GE. These two computed lengths should "balance", that is, be equal. If these lines are equal, the other sides (BC, CF, etc.) may then be figured from any determined line. However, in this case these two lengths for FE are not equal, and must be balanced before other lines can be figured. This balancing may be done by either one of two methods, namely, changing slightly the measured lines or accepting the mean of the two figured lengths for a given line and using this mean in figuring the lengths of intermediate lines. The former method is used when the triangulation control is rectangular in shape and when no one station is enclosed entirely by triangles. The latter method is used when one station is common to a large number of triangles and is enclosed entirely by triangles. In this case the measured line is usually radial from the common station.

When the method of corrected base lines is used, Plate 3, the lines are corrected in proportion to their lengths and these corrected lengths used to figure all other lines. Several corrections may have to be applied before a satisfactory length is found for each of the two base lines. In the case shown on Plate 3 the base line AD was increased 0.05 ft. and line IJ decreased 0.02 ft. This correction is so small that the plotted length will not be changed even slightly. In fact, only in extremely accurate survey work is it necessary to balance these or any other lengths in the triangulation control.

Usually with continuous azimuth and triangulation, the stations are plotted by co-ordinates (latitudes and departures) which has proven satisfactory in regard to both accuracy and simplicity. Ordinary protractor angles and scaled distances, used in plotting stadia side shots, are not sufficiently accurate for plotting stations. In figuring co-ordinates the azimuth readings are converted to bearings. When the azimuth is less than 90° the bearing angle is the same as the azimuth and is in the northeast quadrant. When the azimuth is greater than 90° but less than 180°, the bearing angle is 180° minus the azimuth and is in the southeast quadrant. When the azimuth is greater than 180° but less than 270°, the bearing angle is the azimuth minus 180° and is in the southwest quadrant. When the azimuth is greater than 270° but less than 360° the bearing angle is 360° minus the azimuth and is in the northwest quadrant. Then with these bearings figured the latitudes and departures are figured, using the formula, Cosine of Bearing x distance = Latitude

Sine of Bearing x distance = Departure

Using Plate 3 as an example, the bearing of line AB is N.25°-20'E, which is also the azimuth. The length of line AB is 397.95 ft.

Cosine 25°-20' x 397.95 = Latitude

.903834 x 397.95 = 359.68 N (Latitude)

Sin 25°-20' x 397.95 = Departure

.427884 x 397.95 = 170.28E (Departure)

Thus latitudes and departures for all station lines are figured, closing a complete traverse around the outer periphery. This traverse may be closed continuously in one direction, or in two directions to some common station. As shown on Plate 3, latitudes and departures were figured from A to K through B,C,F,G,H, and J as one unit; and from A to K through D,E, and J as one unit. These composite latitudes and departures from A to K through the two routes should be equal but they usually have a slight difference, due to minor discrepancies in computations. These errors are usually so small, when angles and distances have been figured and balanced correctly, that it is not necessary to balance the co-ordinates. However, balancing is done by inspection, throwing the error into the longest courses.

It is now necessary to convert these corrected latitudes and departures to co-ordinates so that the stations can be plotted, which is done in the following manner. Co-ordinates are assumed for the initial station (station A, North 2000.00 and East 1200.00) in such location that all co-ordinates will be in one quadrant, usually the northeast. Then latitudes "north" are added to the assumed "north" latitudes for this station and latitudes "south" are subtracted. In the same manner departures "east" are added to the assumed "east" departures, and departures "west" are subtracted. In this way the work is kept in the northeast quadrant.

Station B, Plate 3, illustrates the computation of co-ordinates.

From station A the distance is 397.95 ft.and the bearing is N.25°-20' E. The latitude and departure, as determined on page 50 , are 359.68 ft.N. and 170.28 ft. E. Then, 2000.00 (assumed for A) plus 359.68 = 2359.68, north co-ordinate for B.

1200.00 (assumed for A) plus 170.28 = 1370.28 east co-ordinate for B.

The co-ordinates of station B are, therefore, 2359.68 North and 1370.28 East.

The co-ordinates of all stations are figured and kept in tabulated form as shown on Table 2.

VERTICAL CONTROL

Whenever possible the station elevations should be carried by direct rather than by trigonometric levels. The use of vertical arcs adds more work to the office computations as well as additional possibilities of errors.

In a general way, the entire series of elevations throughout the area is based upon the elevations of the station hubs. The first step in securing these elevations is to compute the elevation of some hub located near the bench mark, through the usual process of direct leveling. The next step is to show in sketch form the observed differences in elevation between the various stations of the network. As noted in the chapter on field work these intra-station differences in elevation will have been read in both directions, forward and backward, and upon the sketch should be shown the difference in elevation between each two stations as observed from each direction, as on Plate 4. If the observation was a direct level the difference in elevation will be the difference between the rod reading on the observed station and the height of the telescope axis above the top of the station occupied. In case the observation was by vertical arc the difference in elevation between the telescope axis and the observed point on the rod is secured by the formula, "True horizontal distance multiplied by the tangent of the angle of elevation or depression equals the difference." If the observed station is lower than the occupied station the difference in elevation between the station is equal to the difference in elevation along the line of sight plus the point on the rod to which the vertical arc was observed, minus the height of the telescope above the occupied hub. If the observed station is higher than the occupied station the difference in elevation between the hubs is equal to the difference in elevation along the line of sight plus the height of the telescope axis above the hub and minus the point observed on the rod.

Stations A and D shown on Plate 4 furnish examples. The distance from A to D is 655.65 feet and the vertical arc is $-1^{o}-22'$ read at 4.00 ft. on the rod. The distance of the telescope axis above hub A is 4.65 ft.

655.65 x tangent $-1^{o}-22'$ = Difference in Elevation

655.65 x .02386 = 15.64 ft. (Difference in Elevation along the line of sight)

The observed difference in elevation then from hub A to hub D is

15.64 ft. plus 4.00 ft. (observed on rod) -

4.65 ft. (height of telescope axis above the hub)

= 14.99 ft.

As an example of direct leveling between stations, using the same set-up height of 4.65 ft. at station A, the observed level reading on hub B was 8.27 ft. The difference in elevation then as observed from A, between A and B would be,

$$8.27 \text{ ft.} - 4.65 \text{ Ft.} = 3.62 \text{ ft.}$$

Because of errors in reading vertical arcs, the difference in elevation from one station to another, forward and backward, does not always check. When this is true, it will be necessary to balance these differences, preferably in the same general way that the horizontal angular closures were balanced. This work is also simplified by the use of a sketch on which differences in elevations between stations take the place of azimuths and interior angles, as shown on Plate 2. The procedure, however, is practically the same in both cases. For example (triangle ABD on Plate 6) from station A to station B the difference in elevation is 3.62 ft., B being the lower, and from station B to station A the difference is 3.58 ft. The mean of these two is 3.60 ft. From station A, station D is 14.99 ft. lower. From station D, the difference in elevation is 15.11 ft. This mean is 15.05 ft. Then, from station B, station D is 11.46 ft. lower, and from D the difference to B is 11.52 ft.

The mean of these differences is 11.49 ft. The mean between station A and station D is the greatest and is the total difference in elevation, either from A to D directly or from A to D through station B. Expressed in the form of an equation this would read,

> "Difference in elevation from A to D should equal difference in elevation from A to B plus the difference in elevation from B to D, or, 15.05 ft. should equal 3.60 ft. plus 11.49 ft. but actually equals 15.09 ft."

The error in closure of this triangle therefore is 0.04 ft. Inasmuch as line AD was somewhat obscured the chances are that the greatest error is in this line and the triangle can logically be balanced by inspection as follows,

15.07 ft. = 3.59 ft. plus 11.48 ft.

From the setup at station A the backsight on the bench mark was 2.36 ft., making the height of the instrument 502.36. The measurement of the hub below the instrument was 4.65 ft., therefore the height of hub A is 497.71. According to the new balance hub B is 3.59 ft. below hub A or at elevation 494.12. The height of instrument at B is found by adding to this elevation the height measured to the telescope axis. The difference in elevation between hub B and hub D is 11.49 ft. giving an elevation on hub D of 482.64. In the same way, by adding to this the setup height, the height of instrument at station D is secured. The difference in elevation between stations B and D has now been accepted as correct and cannot be altered when the triangle BCD is balanced. All corrections in this triangle must necessarily be made in the differences in elevation between stations B and C, and D and C.

The same procedure of balancing differences in elevations between stations and computing hub elevations is followed throughout the entire vertical control. It will be seen on Plate 4 that only triangles ABD and BCD were balanced and completed. These two triangles, however, are sufficient and the absence of others makes the sketch much less confusing.

STADIA SIDE SHOTS

Usually there are two distance corrections to be applied to stadia side shots when direct levels are read and three when trigonometric levels are read. The focal corrections, discussed on pages 28-29, is applied in the field. The other two, stadia-hair correction and vertical arc correction, are applied in the office.

The stadia-hair correction is figured on a percentage basis in the field, usually by comparing stadia measurements of intra-station lines with computed distances. This percentage of error is added to or subtracted from the stadia distances as required.

The vertical arc correction is applied only to those side shots in which trigonometric levels were read. The corrected distance is usually figured with a slide rule, but stadia computors and various tables and diagrams are useful. Practically all methods are similar and are based upon the following formula:

Corrected Stadia Distance x 1-(sin.vertical arc)^2

= True Distance.

For example, with the corrected stadia distance assumed at 153 feet, and the vertical arc -7°-12',

$153 \times 1 -(\sin -7^{\circ}\text{-}12')^2$ = True Distance

$153 \times 1 -(.125333)^2 =$

$153 \times 1\ (1-.015708) =$

$153 \times .984292 = 151$ feet (True Distance)

The correction for difference in elevation is applied only where vertical angles are recorded. In order to determine the true elevation it is necessary first to figure the difference in elevation. This correction, like the distance correction, may be figured with tables or calculating machines but the stadia slide rule is most generally used. These two corrections are usually figured jointly in one operation. The slide rule principle for determining the difference in elevation is based upon the formula,

Corrected stadia distance x 1/2 sin 2 (vertical angle)

For example, the same case is used as in the preceding paragraph.

$153 \times 1/2$ Sin. 2 ($-7^{\circ}-12'$) = (Difference in Elevation)

$153 \times 1/2$ Sin.($-14^{\circ}-24'$) =

$153 \times 1/2$ (-.247582) = -19.0 ft. (Difference in Elevation)

As previously stated, the difference in elevation is plus or minus according to the sign of the vertical angle, angles of elevation being plus and angles of depression being minus. The height of the instrument (H.I.) is always plus and the point on the rod at which the vertical angle was taken is always minus. These three figures are added algebraically and this sum is the elevation of the point. The direct level reading to a point is always minus and is subtracted from the H.I. This result is the elevation of the point.

All surveying computations should be kept in a special computation file containing a folder for each survey. At the head of the page should appear the plan number, the name of the client, the exact title of the work, the names of the computer and checker, the date, and the number of the sheet. The work should be arranged neatly and systematically so that every part of the computations can be traced by any one who is familiar with such work. Each important value, each column, etc., should be labeled so that it can be readily identified.

It is very important that all calculations should be checked, not merely at the end of the computation but also at as many intermediate steps as possible. In this way a great waste of time may be prevented and serious errors avoided. One good method of checking is to perform the operations by two independent methods when possible. Very often two men do the computing, one man's work acting as a check on that of the other. Every part of the work from the copying of data out of the notebook to the final results should be done independently. However, the added expense does not often justify this procedure, especially on comparatively rough work, and it may be necessary for one man to check his own work.

As previously stated the possible sources of errors are too numerous to discuss in detail. The main sources, however, are incorrect computations, plotting and inking. In computations errors may be made in distance, either by mechanical errors or by misreading the slide rule, or in elevation by mechanical errors, misreading the slide rule or by incorrect algebraic additions. In plotting, errors may be made in azimuth, distance or elevation. The errors in inking are usually the confusion of tree names and in transcribing elevations.

PLOTTING AND FINISHING

Numerous grades of drawing paper ranging from very cheap "detail" to heavy paper mounted on cloth, called "mounted paper" are used for topographic survey. The best paper available is desirable. It is absurd to attempt to economize on paper when the final plotted survey can last only as long as the paper upon which it is plotted. There is generally a right and a wrong side to all papers, which can be distinguished by the "water-mark"; this will read direct when the right side of the paper is toward the observer. To be satisfactory for use a paper should not have a surface too porous to take ink evenly and should be of a fiber such that after scratching with a knife or rubbing with an ink eraser, the surface will still take ink effectively. For making copies of the map, a very good grade of tracing cloth is desirable.

Topographic conventional signs are used to represent the form of the surface and such physical features as roads, buildings, trees, rivers, etc. For the most part such symbols have been adapted as will suggest by their shape the plan of the object represented and at the same time be readily recognized and easily drawn.

The kind of conventional signs used on any map has an intimate relation to the purposes for which the map is made, therefore, landscape architects' plans and other similar large scale maps require certain changes in the conventional signs which are used on Government surveys and are generally accepted. The more common variations will be described later (pages 67 to 69), under their respective divisions. Whenever it is possible, all conventional signs should be drawn to the actual scale of the map.

HORIZONTAL CONTROL

Before the actual work of plotting has been started, the size and layout of the sheet required for the survey should be determined. A key map similar to Plate 4 is usually used for this work. This map should be drawn to a definite scale but need not be extremely accurate.

Beginning with the initial station, all stations are plotted by means of protracted angles and ordinary scaled distances. Then, from those stations nearest the boundaries, the most distant exterior side shots are plotted in the same way. With this information plotted, the size and orientation of the map can be determined without difficulty, but with assurance that the sheet is large enough for the most distant points. It is often advisable to construct this key map before the co-ordinates of the initial station are assumed. From this map the latitudes and departures can be scaled, approximately only, from the bottom and left sides respectively. These distances are usually increased to the nearest even hundred feet and assumed as the co-ordinates of the initial station.

All stations of the triangulation system are plotted from co-ordinate lines, both latitudes and departures, which are laid out at 10 inch intervals regardless of the size or scale of the map. This interval is adapted mainly because of the ease of scaling between co-ordinate lines with a 12 inch boxwood scale, and because the lines are not too close to make plotting and interpolating more complicated. Extreme care should be exercised in plotting these co-ordinate lines because the correct station locations and finally the accuracy of the plotted results depend upon these lines being plotted accurately. Co-ordinates are usually plotted from one base line from which a perpendicular is erected. These two right angle lines usually represent the extreme westerly and extreme southerly lines. With a boxwood scale, ten inches is measured, on either line, as accurately as is possible. Then, with a beam compass, and using this 10 inch measurement as a radius, the other points of intersecting co-ordinate lines are located by inscribing short arcs from points already determined, and by carefully checking each intersection by direct scaling.

After the co-ordinate lines are laid out and the station co-ordinates figured, the stations are plotted in the following manner. For latitude, along a north and south co-ordinate line, the required distance, total latitude for the particular station is measured. Then, through this point a line is drawn parallel to the east and west co-ordinate line. To secure the departure, the required distance, total departure, is measured along an east and west co-ordinate line. A line, parallel to the north and south lines is drawn through this point. The intersection of these two lines thus constructed is the location of the station.

Intra-station lines are plotted by simply connecting with a straight line those stations that were observed from each respective station. However, the length of each line is carefully checked and when an appreciable error is present the line and the stations which it connects are replotted.

TOPOGRAPHIC READINGS

After all transit stations have been plotted and their positions checked the side shots may be plotted by means of protracted angles and scaled distances. A convenient arrangement for the plotting is as follows. The central portion of a full circle protracted of twelve to fifteen inch diameter is cut out. This protractor must be placed in position by means of two right angle lines parallel to the co-ordinate lines. To the zero end of an engineer's triangular boxwood scale is attached a small piece of mending tape and the scale is secured over the station by inserting a needle through this tape, at zero on the scale, and the station. In this way the zero of the scale is held directly over the point but the scale is free to revolve around the station. The edge of the scale will then be on a radial line, its zero point being at the center.

In plotting a point the scale is turned to read the proper azimuth and the point is then marked by making a pencil dot at the proper reading on the scale. By this method no time is wasted in marking azimuths and in placing and removing the scale.

The plotting of stadia notes requires reading so many notes and scaling so many angles and distances that the work can be done most economically by two men working together, one reading the notes while the other man plots them. The man who is reading the notes should watch the plotting to detect any mistakes and to see that the notes are properly interpreted. Notes are usually read from left to right as they are shown in the level book (Plate 1), but any method satisfactory to the person plotting the survey may be used. Most points are designated by small crosses which also serve as decimals in the elevations of the points. Trees, poles, fire hydrants, etc. are usually shown by a dot, the exact location of the point, enclosed by a circle. When plotting such points it is advisable to record the description before the elevation, instead of the usual method of elevation before description. When this is done the person plotting can determine immediately if the point should be designated by a cross or by a dot.

Referenced points may be plotted either before or after the description and the elevation for the initial point has been recorded. However, elevations for referenced points are stated as being a certain distance higher or lower than the initial point; therefore, the elevation for the initial point is usually recorded before the referenced points are plotted. It should be remembered that the words "right" and "left" refer to the relative position of the referenced point, observed from the station to the initial point and not from the initial point to the station.

A topographic map should be finished in such a manner that it will convey the desired information and can readily be interpreted. The extensive use of color is not recommended but it is necessary to use a limited amount of color in order to distinguish more readily between the different features. These colors are applied only in the form of ink lines and should be consistent in all surveys made by one organization. This color scheme may be any one of several generally used or it may be an original one adapted to fit the conditions at hand. The color scheme discussed herein is not universal but it has proven very satisfactory in several landscape architects' offices.

The co-ordinates and traverse control should be inked immediately after the stations and intra-station lines are plotted and checked. Co-ordinate lines are left uninked until the stations are plotted because inking usually destroys a certain degree of the accuracy with which they are plotted. These lines are inked with a very fine green line. For the traverse control red ink is used. The stations are shown as triangles with a black dot in the centers, representing the exact point of the station. Intra-station lines are inked from the sides of the triangles and not from the exact points of the stations. No other inking is done until the entire survey has been plotted.

After the plotting is completed, buildings, trees, and all other physical features such as roads, fences, etc., are inked in the order named. Trees are represented by black dots drawn, if possible, to the scale of the plan. The diameter and the name of the tree are shown above the dot and the elevation below. Usually, foliage lines are not shown except in orchards where the outline is indicated, and mass foliage outlines where only the outstanding trees are located.

Buildings and all other physical features are also shown in black ink. When the building is of sufficient importance and when the plan is drawn to a scale which permits it, fenestration is usually shown. Where any doubt exists it is always advisable to show fenestration.

Elevations are shown in red ink just as they were plotted, that is, with a small cross indicating the decimal and the point when the point is one of elevation only, and under the point when the point is some feature such as a tree, fire hydrant, or pole.

In all lettering on topographic maps simple rather than elaborate and ornamental styles are preferable. Black ink is always used for lettering which should be done after all the physical features are inked but before the contours are inked. In this way no important points will be obliterated and the lettering will not be crowded between contours.

The interval or vertical distance between contours depends mainly upon the scale of the map and the particular use for which it is being made. Also, the diversity of the terrain, the size of the area surveyed, the total difference in elevation, and the time allotted for the work are determining factors. For general landscape work an interval of one foot is most desirable; however, intervals of two, five and even ten feet are sometimes used.

Contours are usually interpolated by estimating (by eye) or by the method of similar triangles. The former method requires great skill and an appreciable knowledge of physiography. The latter method does not allow for any variations not shown directly by elevations. It has the advantages of being one of the least involved and most accurate method. A combination of the two, however, is even more accurate and better suited to general conditions.

Interpolating contours by the method of similar triangles is merely a mechanical interpolation between two given points of elevation and is accomplished in the following manner. Any two points of known elevation are connected by a pencilled straight line. From the lower of the two and along a line making an angle of approximately 45° with the line between these points, there is laid off, on any convenient scale, a distance equal to the difference in elevation between the two points under consideration. On this line are marked scaled distances equal to the difference in elevation between the lowest and each of the intermediate contours. From the farthest distance so marked, a straight line is drawn to the higher of the original points. Through each of the intermediate points laid off, lines parallel to the last line are drawn to intersect the line between the two original points. Those intersections will be the locations of contours between the original points.

The majority of the contours shown on Plate 7 were drawn by estimating their location between points of known elevation. On the northeast corner of the property, however, contours between 385 and 420 were interpolated by the method of similar triangles.

Contours are usually inked free-hand with a broken line. Every fifth contour, 5', 10', 25' or 50', according to the interval used, is shown in black dashes, or short lines, about one-half inch long. All other contours are shown by orange dashes about half the length of the others and somewhat lighter in weight.

Every finished topographic map should have a suitable title, a scale, a border line, a north point and a note or legend giving any other useful data. All of these features are, of course, shown in black ink, and should be placed in such positions on the map so that the drawing as a whole will look well balanced. These positions will naturally depend upon the spaces around the drawing which are available for this purpose. The weight of lines used should be consistent with the general style of the map.

The title should be so arranged and the size of the letters so chosen that the most important part of the title strikes the eye first. In general, each line of lettering should be centered, and the spacing between the lines should be so arranged that no part will either appear crowded or seem to be "floating away" from the rest of the title. The general outline of the title should be pleasing to the eye. The use of stamped titles has proven to be generally satisfactory from the standpoint of economy and uniformity. These titles are made to suit the particular office but all of them are similar. Any such title is satisfactory provided that it allows space for the name of the client, the description of the plan, the date, scale, and plan number, the initials of those who are responsible for the plan and the name of the organization by which the plan was made.

Every finished topographic survey should contain a scale stated in two ways, namely, (1) by a graphic scale, and (2) by a definite statement as scale: 1" = 40'. The definitely stated scale should be a part of the title or should, like the graphic scale, be shown immediately under the title.

The border line and north point should be consistent in simplicity and in the weight of lines, with the plan. They should not be so heavy or so ornate that they are conspicuous. Notes or legends should be executed in plain lettering like the Reinhardt alphabet, or in simple Gothic lettering. Care should be exercised in placing the legend so that it is definitely subjective to the title.

Views are shown by radial lines from the station from which they were recorded. Those lines show the limits of the views as well as the direction. A short description, as "distant view across valley" should be written along one or both of these lines, or between the two lines which define the view.

It is necessary, of course, that the members of the office party which plot and ink the survey be observant at all times, and maintain the accuracy of the work to as high a standard as conditions will allow. Each person connected with the work should be particularly careful throughout the work. The two main things to watch especially are the orientation and location of the protractor and the scale. Such mistakes as orienting the protractor 180° off, so that 0° or 360° is where 180° should be, and such as locating the protractor so that the station point is not in the exact center seem to be very obvious, but nevertheless they are often made. Also, the use of the 30 or 50 scale when the 40 scale should have been used is often detected. Any one of these mistakes will, of course, destroy the plan from the standpoint of true values or accuracy.

On all plotting work, just as on all field work and computations, frequent checks should be applied to insure accuracy. The traverse which has already been figured to close should now close on the paper. If it does not the error is one of scaling and the entire triangulation should be checked, and if necessary replotted. There is no definite method of checking the plotted side shots. For this reason care should be exercised in plotting them correctly the first time. In no case, however, should a draftsman allow a plan to leave his hands which has not been properly checked and known to be correct.

All of the plotting and inking up to this point has been concerned only with the mounted paper plan which should never be removed from the office, except when it is checked in the field, and from which only photographic prints can be made. It is advisable to make copies of this map from which blue prints may be made, and so that a copy or a print may be sent to the client. Also the copy is generally used as the basis for preliminary studies, etc., where extreme accuracy is not necessary. It will be found, however, that in all future office work connected with the survey the engineer prefers to use the original paper plan rather than the tracing. All tracings of the survey should be made on a very good grade of linen tracing cloth. Plate 8 is a finished tracing of a survey. It will be noted that the tracing contains all the information except co-ordinate lines and elevations of side shots shown on the finished paper plan, Plate 7.

THE COST OF TOPOGRAPHIC SURVEYS

The conditions determining the cost of topographic surveys are so numerous and varied that no rule or formula can be offered whereby the cost can be estimated prior to making the survey. Inspection and comparison with other surveys of similar size and conditions is the only method of estimating the cost, even approximately. The most important determining factors are as follows: (1) the class (city lot, small estate, etc.), (2) the existing development, (3) the vegetation, and (4) the terrain. Also, the experience of the field party, the season of the year, and the weather conditions must be considered. From experience, it has been found that the cost of topographic surveys, made by the triangulation method and using stadia and transit varies according to the size of the area surveyed. The percentage of field work is greater and the percentage of office work is less on larger surveys. For surveys up to 100 acres the field work is approximately 40% of the total time required. This percentage increases up to 200 acres and from 200 acres to 1000 acres the field work is approximately 65% of the total time required. It must be remembered that this is an approximation only and will vary according to conditions, as is shown in the accompanying tabulation.

In order to show the relative costs of topographic surveys of various sizes and conditions a tabulation is given on which all time is converted into transitman hours based upon,

Experienced Transitman - as 1.00

Experienced Rodmans - as 0.75

Common Laborer - as 0.50

The stated conditions are based upon the following classification:

CLASS

City Lots

Suburban Large Lots

Small Estates

Institutions

Large Estates

Subdivisions

Town Sites

DEVELOPMENT

Highly Developed ------------------ A

Developed ------------------------- B

Natural --------------------------- C

VEGETATION

Thick Brush ----------------------- 1

Open Woods - All Trees ------------ 2

Open Woods - Essential Trees ------ 3

Open Woods - Edge Location -------- 4

Scattered Trees ------------------- 5

Entirely Clear -------------------- 6

TERRAIN

Very Steep Slopes --------------- a

Moderate Slopes ----------------- b

Gentle Slopes ------------------- c

Flat ---------------------------- d

CLASS	ACRES	SCALE	VERTICAL INTERVAL	DIFFERENCE IN ELEVATION	DEVELOPMENT	VEGETATION	TERRAIN	FIELD TIME (HRS. PER ACRE)				% FIELD WORK	OFFICE TIME (HRS. PER ACRE)				% OFFICE WORK	TOTAL TRANSIT-MAN HRS. PER A.
								TRANSIT-MAN	RODMAN	COMMON LABOR	TRANSITMAN HRS. EQUIV.		TRANSIT-MAN	RODMAN	COMMON LABOR	TRANSITMAN HRS. EQUIV.		
CITY LOTS																		
1	2	20	1	10	A	2	b	15.00	15.00	2.00	27.25	38.4	25.00	25.00		43.75	61.6	71.00
2	0.58	10	1	10	B	2	d	8.55	6.84		13.68	27.1	36.75			36.75	72.9	50.43
3	1.5	10	1	12	C	1	c	4.00	4.00		7.00	24.5	12.00	12.66		21.50	75.5	28.50
SUBURBAN LARGE LOTS																		
4	3.9	10	1	18	C	2	c	4.62	9.23		11.54	27.7	17.70	16.49		30.07	72.3	41.61
5	6.6	20	1	13	A	2	c	3.33	6.51		8.21	41.6	1.89	12.88		11.55	58.4	19.76
SMALL ESTATES																		
6	10.5	20	1	250	A	5	a	6.50			6.50	27.1	17.50			17.50	72.9	24.00
7	15	20	1	40	A	2	b	2.13	2.13		3.73	25.8	6.12	6.12		10.71	74.2	14.44
8	14	20	1	50	C	2	b	3.16			3.16	35.8	5.68			5.68	64.2	8.84
INSTITUTIONS																		
9	39	30	1	100	B	2	b	1.57	1.57	0.44	2.97	33.1	6.02			6.02	66.9	8.99
10	77	50	1	15	B	5	d	0.82		0.24	0.94	29.9	2.21			2.21	70.1	3.15
LARGE ESTATES																		
(John Miley) 11	26	40	1	140	B	3	b	1.39	2.42		3.21	28.5	3.33	6.26		8.03	71.5	11.23
12	47	40	1	105	B	5	b	1.33	2.00	1.65	3.66	42.4	1.85	4.16		4.97	57.6	8.63
13	120	60	1+5	120	B	3	a	1.34	0.70		1.87	42.3	2.06	0.65		2.56	57.7	4.41
14	100	60	2	200	B	3	a	1.44	0.72		1.98	45.2	2.09	0.41		2.40	54.8	4.38
15	47	40	2	96	B	4	b	0.77	0.96	0.77	1.87	44.5	1.15	1.57		2.33	55.5	4.19
SUBDIVISIONS																		
16	170	100	5	150	B	5	b	0.60			0.60	42.3	0.82			0.82	57.7	1.42
17	176	200	5	100	C	6	b	0.32	0.77		0.90	79.1	0.13	0.14		0.24	20.9	1.13
18	405	200	2	40	B	4	c	0.15	0.08	0.38	0.39	69.7	0.14	0.03		0.17	30.3	0.57
TOWN SITES																		
19	840	200	2	125	C	1	b	0.78	1.26		1.73	62.2	0.78			1.05	37.8	2.78

COST OF TOPOGRAPHIC SURVEYS

A TOPOGRAPHIC SURVEY FOR JOHN N. MILEY
CINCINNATI, OHIO

A specific instance of the application of the methods and principles herein described was found in the above mentioned survey.

The problem was to make a topographic survey of an estate of twenty-six acres to be used by a landscape architect in his development of this estate. Such a survey should include all information which may be of importance to the landscape architect. The method of making this survey should be one which provides for accuracy as well as for economy in both field and office work. Further, this method should be one which is commonly used so that it may be elaborated or used on adjoining territory by someone else, should the occasion demand it.

The John N. Miley Survey shows typical conditions encountered in the making of topographic surveys and in the landscape development of country estates in its general vicinity. This particular survey was selected for this reason, and because the field work was done under very adverse weather conditions which naturally affected the accuracy of the survey, thus making balancing necessary in each step of the office computations. A deliberate attempt was made to secure such a survey in preference to one with which less difficulty was encountered. Therefore, the problem is a true representation of practical conditions, and not a theoretical representation.

Plate 8 shows the topographic description of this property. The estate of Mr. John N. Miley is situated on a secondary through highway about twenty miles from the center of the city. It consists of twenty-six acres about half of which is open and the remainder of which is in open woods. The terrain is irregular and slopes to the east and to the creek at the eastern end of the property. The western end consists of gentle slopes while the eastern end is moderate with some steep slopes. There is no development on this end of the property. An old farmstead and orchard is situated on the western portion. The development taken as a whole is only slight. The natural growth is sparse with very few outstanding trees and a negligible amount of undergrowth, having been formerly used as a pasture and the headquarters, at the western end, for a larger estate. The existing buildings are all in excellent condition and will probably be preserved as the central part of the service area. There are no outstanding views except one toward the east across the creek and the country club grounds. The soil is generally poor; the eastern end is very badly washed and in need of organic matter, the western end has been under cultivation and is in fair condition. The total difference in elevation is 140 feet.

The method of procedure as previously explained in detail in Part One was followed throughout the development of this survey. Only the variations from this stated procedure are noted herein.

The field party of three consisted of an experienced transitman, a trained rodman who acted as notekeeper and who guided the third member, an untrained assistant. An ordinary Buff transit which read stadia one-half of 1% high was used.

The method of horizontal and vertical control as explained was adapted, measuring directly lines AD and IJ, and using an assumed bench (Plate 8) as the datum for all elevations. Line AD was measured on a level while line IJ was measured and figured by the slope distance method. The temperature correction was not applied to either measurement.

After plotting was completed an additional visit to the property was made by the notekeeper for the purpose of checking by inspection the plotted results of the field work.

All of the office work was executed in the headquarters office by the transitman who computed and read notes, and the notekeeper who plotted. The survey was finally inked and traced by these two men working together.

A Monroe six figure calculating machine was used in connection with the control computations and all stadia reductions were made on the stadia slide rule. The previously described method of computing and balancing was used throughout the computations.

The map was plotted on mounted paper at a scale of 1 inch to 40 feet with a vertical interval of 1 foot, and finished in accordance with the recommended practice. The map was checked very carefully and finally traced, on cloth, so that blueprints could be made.

The cost of this survey as well as all others tabulated on page 76-a is given in hours only. If the cost were given in money the information would be of value in only one location and only one season or year. Salaries, of course, vary according to economic conditions but the efficiency of experienced men qualified to do topographic surveying is to a certain extent standard. At least it is more uniform than economic conditions.

The total converted transitman hours per acre was 11.23. It will be seen that this survey was the most expensive in its particular class. There are two main reasons for this, namely, (1) adverse weather conditions under which the field work was done, thereby increasing the amount of office work in balancing and closing the control of the survey; and (2) the size of the survey. This survey is the smallest in its class and more nearly approaches a small estate, and the smaller the area the greater the number of hours per acre, all other conditions being equal.

For field work the number of converted transitman hours were 3.21 per acre for the preliminary work, establishing control, and for the final work, recording topographic information. For office work the number of hours per acre was 8.03, distributed among the different items of computation, plotting, and inking and finishing but not the tracing which is considered separately. These divisions of field work and office work are so closely related that it would be exceedingly difficult further to analyze the cost, unless it were kept separately at the time of executing the work. On a percentage basis the field work required 28.5 per cent and the office work 71.5 per cent.

All plans are presented in the appendix in the form of photographic reductions. The size and bulkiness of the originals prohibit their being incorporated herein. Each step in the development of the final tracing of the finished topographic map was drawn separately, the reproductions of which are shown. While the originals were plotted and drawn with extreme care and accuracy it must be remembered that the photographic reductions are to a certain degree inaccurate and cannot be scaled directly.

ACKNOWLEDGEMENTS

For permission to use the survey described herein the author is indebted to A. D. Taylor, Landscape Architect of Cleveland, Ohio. For very helpful assistance, criticisms and suggestions for improvements, much credit is due to the individual members of Mr. Taylor's organization, and to Mr. H. E. Leonard, C.E., of Cleveland, Ohio.

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APPENDIX

INDEX TO ILLUSTRATIONS

VERTEX	SIDE	READ ANGLE	COR.	BALANCED ANGLE	FIG.	CLOSURE +	CLOSURE −	COR.
A	B-D	70°-53'	+30"	70°-53'-30"	ABD		60"	+60"
B	A-D	73°-31'	✓	73°-31'	BCD		60"	+60"
	C-D	42°-44'+	+30"	42°-44'-30"	CDF		30"	+30"
C	B-D	94°-23'	+30"	94°-23'-30"	DEF		30"	+30"
	D-F	69°-08'-30"	+30"	69°-09'	EFG		30"	+30"
D	A-B	35°-35'	+30"	35°-35'-30"	EGH	60"		-60"
	B-C	42°-52'+	✓	42°-52'	EGI	60"		-60
	C-F	50°-49'	✓	50°-49'	EHJ			✓
	E-F	47°-48'	+30"	47°-48'-30"	GHI		30"	+30"
E	D-F	36°-48'-30"	✓	36°-48'-30"	HIJ			✓
	F-G	50°-54'-30"	✓	50°-54'-30"	IJK			✓
	G-H	24°-34'	-30"	24°-33'-30"				
	G-I	61°-02'	-30"	61°-01'-30"				
	H-J	45°-22'	✓	45°-22'				
F	C-D	60°-02'	✓	60°-02'				
	D-E	95°-23'	✓	95°-23'				
	E-G	30°-00'-30"	+30"	30°-01'				
G	E-F	99°-04'-30"	✓	99°-04'-30"				
	E-I	55°-48'	✓	55°-48'				
	E-H	112°-10'	-30"	112°-09'-30"				
	H-I	56°-11'-30"	✓	56°-11'-30"				
H	E-G	43°-17'	✓	43°-17'				
	E-J	77°-09'	✓	77°-09'				
	G-I	85°-32'	✓	85°-32'				
	I-J	34°-54'	✓	34°-54'				
	G-E	63°-11'	-30"	63°-10'-30"				
I	G-H	38°-16'	+30"	38°-16'-30"				
	H-J	100°-36'	✓	100°-36'				
	J-K	41°-16'-30"	✓	41°-16'-30"				
J	H-I	44°-30'	✓	44°-30'				
	E-H	57°-29'	✓	57°-29'				
	I-K	100°-59'	✓	100°-59'				
K	I-J	37°-44'-30"	✓	37°-44'-30"				

TABLE - 1

BALANCED HORIZONTAL ANGULAR CONTROL

COURSE	AZIMUTH	BEARING	COS.	SIN.	DIST.	LATITUDE N	LATITUDE S	DEPARTURE E	DEPARTURE W	CO-ORDINATES N	CO-ORDINATES E
A						+.01				2000.00	1200.00
A-B	25°-20'	N 25°-20'E	.903834	.427884	397.95	359.68		170.28		2359.69	1370.28
B-C	89°-04'-30"	N.89°-04'-30"E	.016144	.999870	440.82	7.12		440.76		2366.81	1811.04
C-F	105°-32'	S.74°-28'E	.267799	.963475	393.49		105.38	379.12		2261.43	2190.16
F-G	100°-06'	S.79°-54'E	.175367	.984503	461.09		80.86	+.02 453.94		2180.57	2644.12
G-H	68°-52'	N.68°-52'E	.360540	.932744	180.10	64.93		167.99		2245.50	2812.11
H-I	163°-20'	S.16°-40'E	.957990	.286803	241.58		231.43	69.29		2014.07	2881.40
I-K	42°-39'-30"	N.42°-39'-30"E	.735408	.677625	316.27	232.59		214.31			
						664.32	417.67	1895.69			
						417.67					
A						246.65				2000.00	1200.00
A-D	96°-13'-30"	S.83°-46'-30"E	.108433	.994104	655.65		71.09	651.78		1928.91	1851.78
D-E	93°-18'-30"	S.86°-41'-30"E	.057709	.998333	788.31		45.49	−.02 787.00		1883.42	2638.76
E-J	70°-57'	N.70°-57'E	.326393	.945234	464.16	151.50		438.74		2034.92	3077.50
J-K	4°-55'	N.4°-55'E	.996320	.085707	212.53	211.74		18.21		2246.66	3095.71
						363.24	116.58	1895.73			
						116.58					
H						246.66				2014.07	2881.40
H-Mon.	309°-12'	N.50°-48'W	.632029	.774944	193.67	122.41			150.08	2136.48	2731.32

TABLE - 2

TABLE OF COMPUTED CO-ORDINATES

JOHN N. MILEY CINCINNATI, O. Nov. 30, 1925 H.E.B. ⊼ 1

TOPO SURVEY Cloudy K.B.S.

AT △A OLD BUFF GUN Stadia Reads ½ of 1% High C.R.B

AZ.	DIST	LEVEL	VERT.	COR.	ELEV.	DESCRIPTION	HUB - 497.71
0-0						Mag. North	H.I. - 502.36
25°-20'	399	8.27				△ B	H.I. - 4.65
96-13	657.1		-1°-22 (4			△ D (Taped - 655.60)	
		4.65				△ A	
292-41-30						Permanent Backsight - East Gable House	
		2.36			500.00	B.M. Nail in S. Root 30" M.	
334-16	30	2.9			499.5	30" M. B.M. Tree	
245-02	~~102.5~~ 101.5		-1°-08 M.T.	2.0	495.7	15" H.M.	
259-52	~~107~~ 106	6.2			496.2	℄ 12' Rd.ₓ 10'E + 1'↓ is Gutter	
251-10	75	6.6			495.8	10" Cat.	
297-54	88	4.2			498.2	24" chry	
311-56	75	3.1			499.3	15" "	
320-41	85	2.7			499.7	24" "	
307-35	~~141~~ 140	4.1			498.3	Fire Hydr.ₓ 13'E is ℄ Rd.	
314-19	~~137~~ 136	3.4			499.0	℄ Rd at Crest	
321-32	~~133~~ 132		+1-35 (7	3.6	499.0	El. Lt. Poleₓ ℄ Rd. is 12" W.	
323-35	~~120~~ 119		+0-43 M.T.	1.5	499.2	8" chry	
322-35	~~138~~ 137		+1-27 (7	3.5	498.9	⊙ₓ 5' rt. is 8" M.	
323-40	53	2.6			499.8	S.W. Cor. House	
01-19	40	2.5			499.9	S.E. Cor. Porchₓ 5'S. + 4'W. is Wellₓ 5'S. + 6"E is Well	
257-12	26	4.8			497.6	20' Cat.	

PLATE 1 - TYPICAL PAGE FROM NOTEBOOK

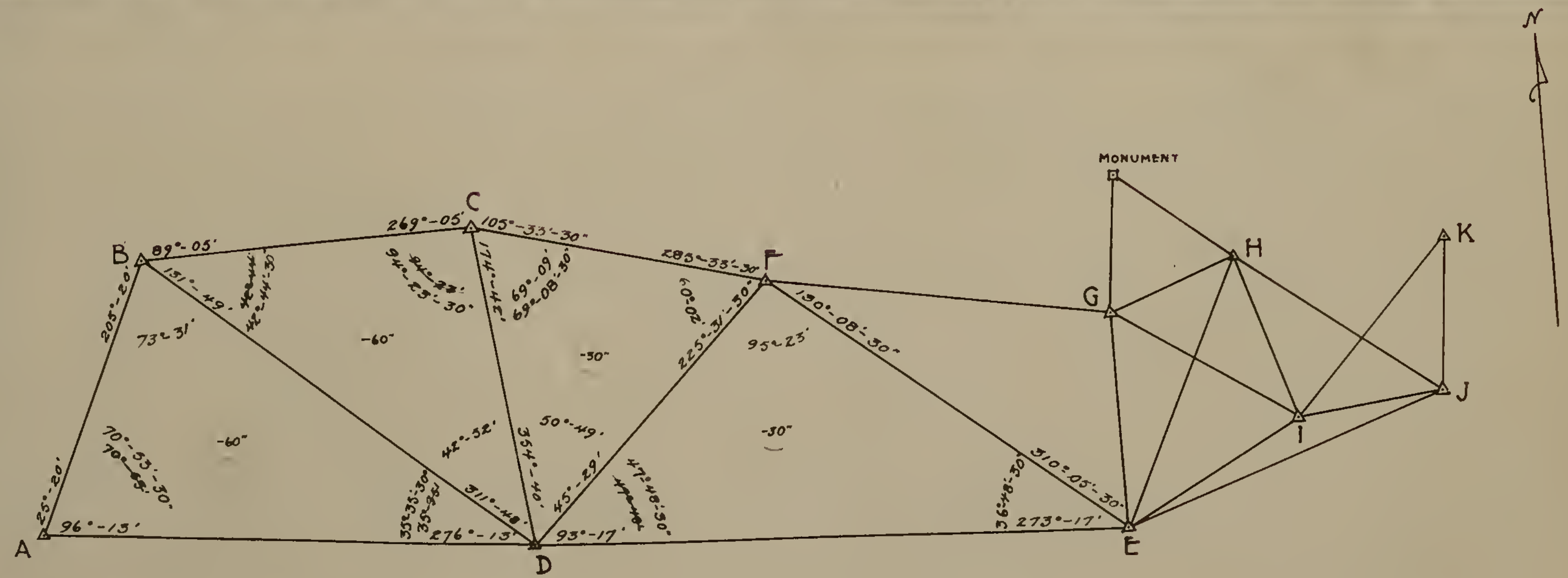
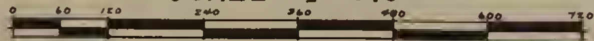


PLATE 2

SKETCH SHOWING BALANCED HORIZONTAL ANGULAR CLOSURES

SCALE – 1" - 240'



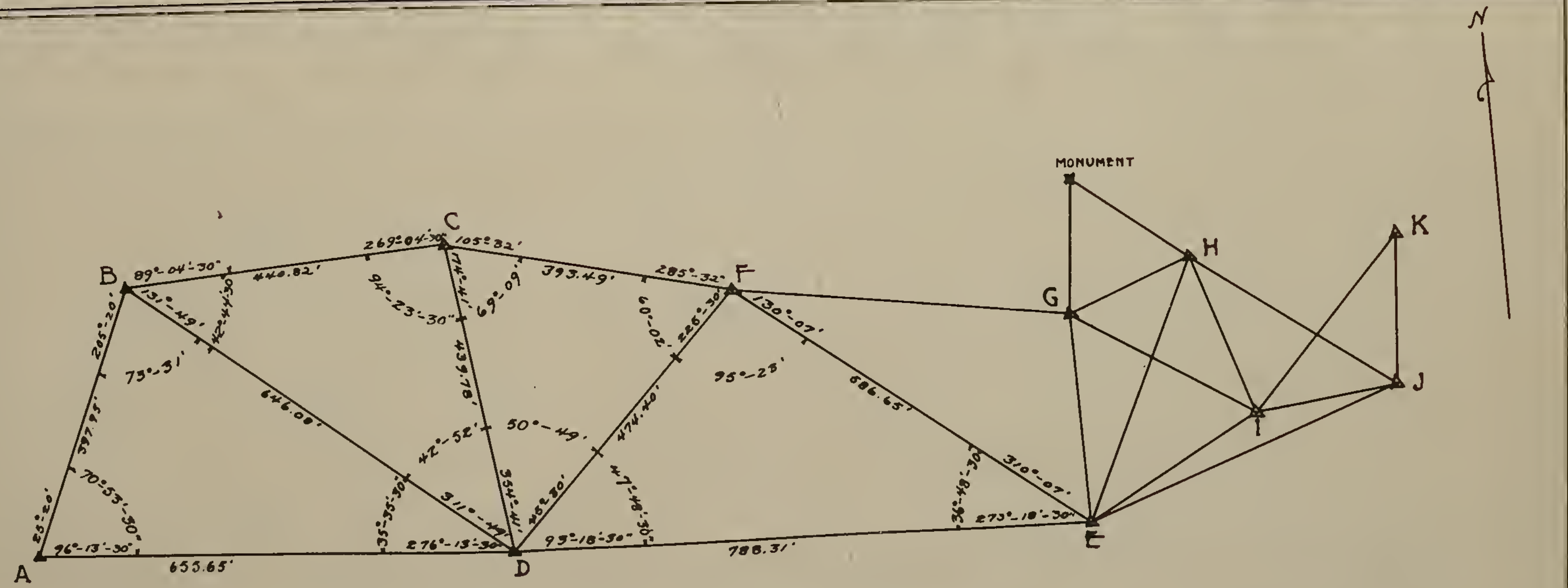


PLATE 3

SKETCH SHOWING COMPUTED INTRA-STATION LINES

SCALE - 1" = 240'

0 60 120 240 360 480 600 720

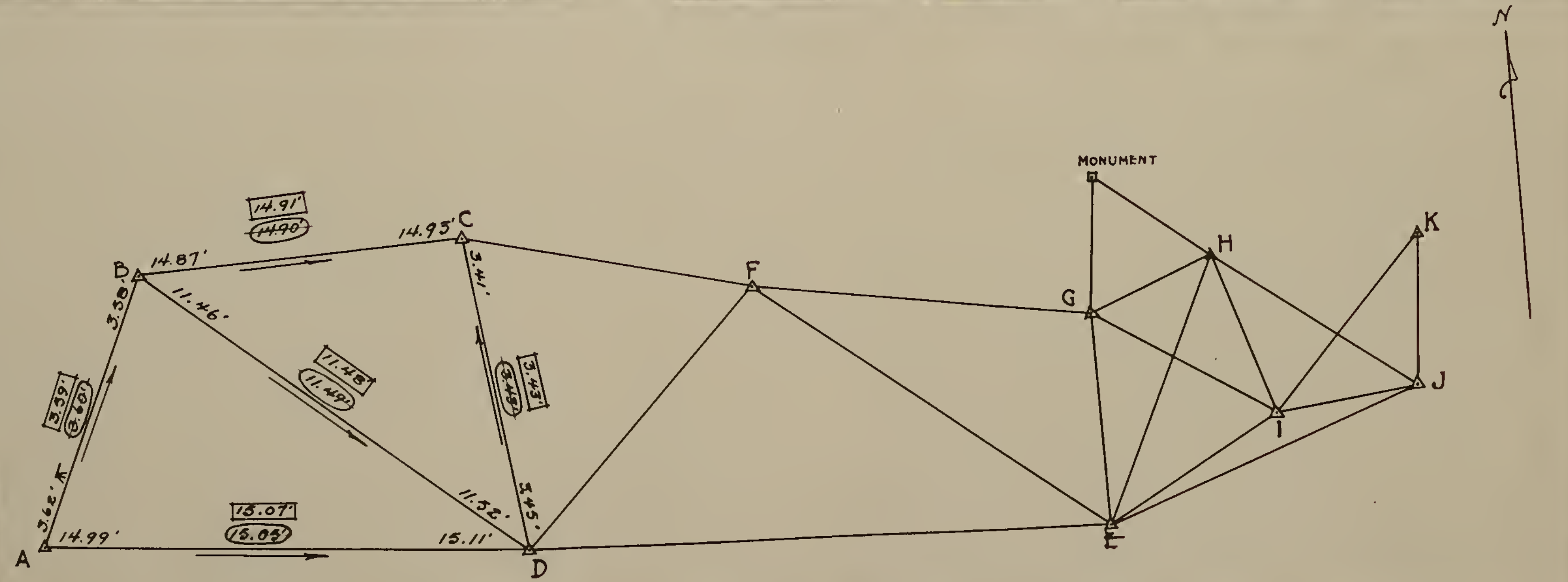


PLATE 4
SKETCH SHOWING BALANCED
VERTICAL CLOSURES
SCALE ~ 1" = 240'

PLATE 5 - PLAN SHOWING PLOTTED HORIZONTAL CONTROL.

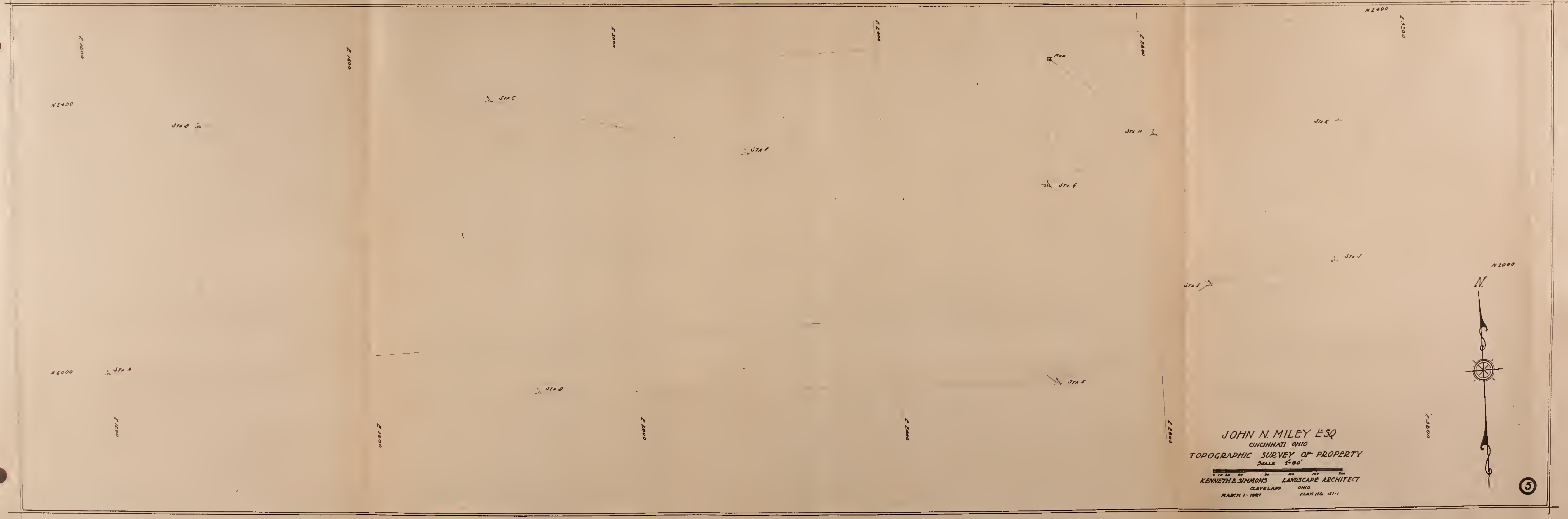

N 2400
N 2000
STA A
STA B
STA C
STA D
STA F
STA G
STA H
STA I
STA J
STA K
N.
JOHN N. MILEY ESQ
CINCINNATI OHIO
TOPOGRAPHIC SURVEY OF PROPERTY
SCALE 1"=80'
KENNETH B. SIMMONS LANDSCAPE ARCHITECT
CLEVELAND OHIO
MARCH 1 - 1927
PLAN NO. 121-1
5

PLATE 6 - PLAN SHOWING PLOTTED DETAILED TOPOGRAPHIC INFORMATION.

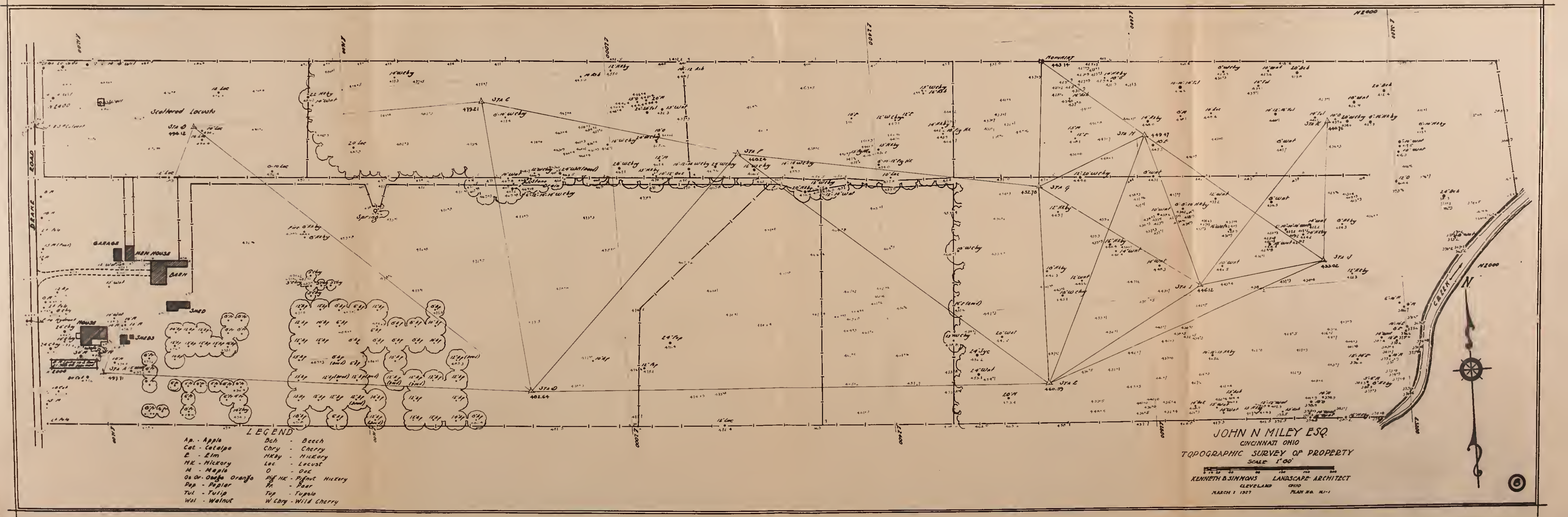
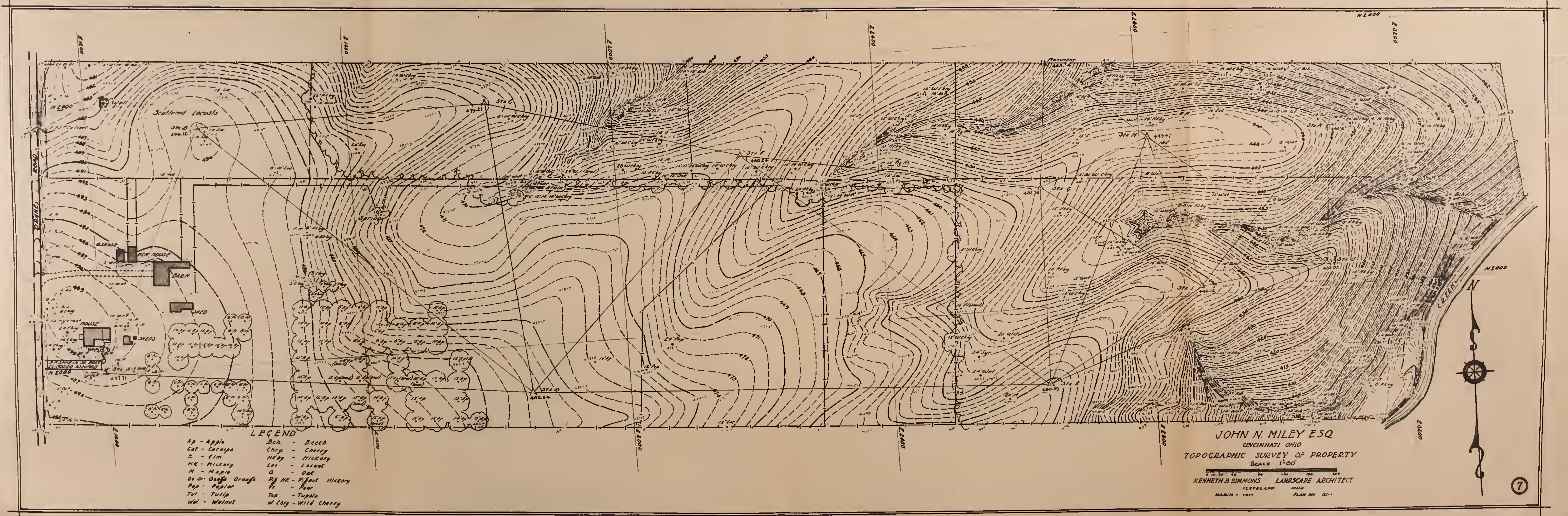

JOHN N MILEY ESQ.
CINCINNATI OHIO
TOPOGRAPHIC SURVEY OF PROPERTY
SCALE 1"60'
KENNETH B. SIMMONS LANDSCAPE ARCHITECT
CLEVELAND OHIO
MARCH 1 1927
PLAN NO. MJ-1
LEGEND
Ap. - Apple
Cat. - Catalpa
E. - Elm
HK. - Hickory
M - Maple
Os Or. - Osage Orange
Pop. - Poplar
Tul - Tulip
Wal - Walnut
Bch. - Beech
Chry - Cherry
HKby - Hickory
Loc. - Locust
O - Oak
Pig HK - Pignut Hickory
Pr. - Pear
Tup - Tupelo
W Chry - Wild Cherry
DRAKE ROAD
GARAGE
HEN HOUSE
BARN
SHED
HOUSE
SHEDS
Spring
Scattered Locusts
CREEK
N

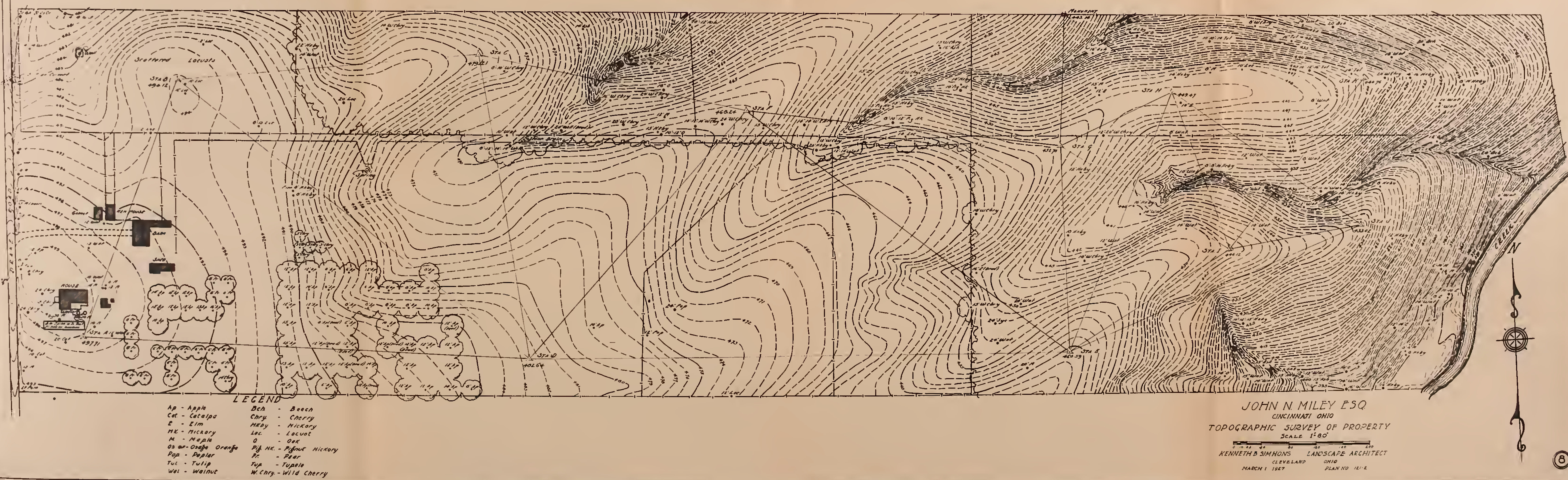
PLATE 7 - PLAN SHOWING COMPLETED PAPER MAP.



JOHN N. MILEY ESQ.
CINCINNATI OHIO
TOPOGRAPHIC SURVEY OF PROPERTY
SCALE 1"=60'
KENNETH B SIMMONS LANDSCAPE ARCHITECT
CLEVELAND OHIO
LEGEND
Ap - Apple
Cat - Catalpa
E - Elm
HK - Hickory
M - Maple
Os Or - Osage Orange
Pop - Poplar
Tul - Tulip
Wal - Walnut
Bch - Beech
Chry - Cherry
HKby - Hickory
Loc - Locust
O - Oak
Pig HK - Pignut Hickory
Pr - Pear
Tup - Tupelo
W. Chry - Wild Cherry
Scattered Locusts
Spring
Monument
BARN
HOUSE
SHED
SHEDS
GARAGE
CREEK
N

7

PLATE 8 - PLAN SHOWING FINAL TRACING.


Scattered Locusts
HOUSE
CREEK
N
LEGEND
Ap - Apple
Cat - Catalpa
E - Elm
HK - Hickory
M - Maple
Os or - Osage Orange
Pop - Poplar
Tul - Tulip
Wal - Walnut
Bch - Beech
Chry - Cherry
HKby - Hickory
Loc - Locust
O - Oak
Pig HK. - Pignut Hickory
Pr - Pear
Tup - Tupelo
W. Chry. - Wild Cherry
JOHN N. MILEY ESQ
CINCINNATI OHIO
TOPOGRAPHIC SURVEY OF PROPERTY
SCALE 1"=80'
KENNETH B SIMMONS LANDSCAPE ARCHITECT
CLEVELAND OHIO
MARCH 1 1927
PLAN NO 121-2

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