Chapter 4
Topographic Survey Techniques

4-1. General

This chapter outlines the most common field techniques used in performing topographic surveys. The primary focus is on electronic plane table methods and electronic total station techniques. Transit station methods are rarely used and they are not covered. Photogrammetric methods of acquiring topographic data are covered in EM 1110-1-1000. Kinematic GPS topographic surveying methods are covered in EM 1110-1-1003. GPS and total station equipment usage is the best combination for topographic map production. This chapter focuses on topographic surveying techniques used for detailing large-scale site plan maps for engineering and construction.

4-2. Engineering Site Plan Surveys

An engineering site plan survey is a topographic (and, if necessary, hydrographic) survey from which a project is conceived, justified, designed, and built. The methods used in performing an engineering survey can and sometimes will involve all of the equipment and techniques available. Photogrammetry may be used to produce maps of almost any scale and corresponding contour interval. Profiles and cross sections may also be obtained from aerial photos. The accuracy of the photogrammetric product varies with the contour interval and must be considered when planning such a project.

a. The plane table and alidade may be used to produce maps in the field. A blank map upon which control points and grid ticks have been plotted is mounted on the plane table. The table is mounted on a low tripod with a specially made head. The head swivels so that it can be leveled, locked in the level position, and then be rotated so that the base map can be oriented. The base map is a scaled plot of the ground control stations. Thus, with the table set up over one of the stations, it can be rotated so that the plotted stations lie in their true orientation relative to the points on the ground. Spot elevations and located features are usually located with an alidade which uses stadia to determine distance. The error of a map produced with a plane table and alidade varies across the map as the error in stadia measurements varies with distance. Horizontal errors may range from 0.2 foot at 300 feet to 10 feet or more at 1,000 feet. Since the elevation of the point is determined from the stadia measurement, relative errors in the vertical result.

b. Transit tape topographic surveys can be used to locate points from which a map may be drawn. The method generally requires that all observed data be recorded in a field book and the map plotted in the office. Angles from a known station are measured from another known station or azimuth mark to the point to be located and the distance from the instrument to the point. The elevation of the point is determined by stadia or other means such as chaining or electronic distance measurement (EDM) and differential leveling. The accuracy may be slightly better than the plane table-alidade method or very high (0.1 foot or less) depending upon the equipment combinations used. Transit tape topographic survey methods are rarely employed and are not covered in this manual. Procedures for performing transit tape topographic surveys can be found in older survey texts.

c. If there is extreme congestion, the plane table is often used in combination with transit tape. In this case, points are plotted as they are located. The advantage of the plane table, namely the developing map, at the jobsite does away with the disadvantage of transit tape (angle distance)--the absence of the real view while the map is being produced. The accuracy of the combined method is usually very high as equipment combinations are generally used that yield the best results. Oftentimes theodolite, EDM, and direct leveling are used to locate the point.

d. The last and most commonly employed method is the total station. The remainder of this chapter discusses field topographic survey techniques using plane table and electronic total station methods.

4-3. Utility Surveys

a. *Definition*. Utility surveys can be of several types, but principally they can be divided into two major types. One type is the layout of new systems, and the other type is the location of existing systems. The manual confines the definition of utilities to mean communications lines, electrical lines, and buried pipe systems including gas, sewers, and water lines. The layout of new systems can be described as a specialized type of route surveying, in that they have alignment and profiles and rights-of-ways similar to roads, railroads, canals, etc. In reality utilities are transportation systems in their own right. Utilities are special in that they may have problems regarding right-of-way above or below ground. This section does not cover route layout for utilities since route surveying principles are covered in a following chapter.

b. Uses. A great portion of utility surveying is the location of existing utilities for construction planning, facility alteration, road relocations, and other similar projects. This is a very important part of the preliminary surveys necessary for most of these projects.

c. Techniques. Utilities are usually located for record by tying in their location to a baseline or control traverse. It may be more convenient to locate them with relation to an existing structure, perhaps the one that they serve.

(1) Aboveground utilities are usually easily spotted and are a bit easier to locate than the buried variety. Therefore, they should present little difficulty in being tied to existing surveys. Pole lines are easy to spot and tie in. Consulting with local utility companies before the survey has begun will save much work in the long run. Any plats, plans, maps, and diagrams that can be assembled will make the work easier. If all else fails, the memory of a resident or nearby interested party can be of great help.

(2) Proper identification of utilities sometimes takes an expert, particularly regarding buried pipes. There are many types of wire lines on poles in this modern age of electronics; this also can lead to identification problems. Where once only power and telephone lines were of concern, now cable TV, burglar alarms, and maybe even other wire line types must be considered. Much resourcefulness is required to identify the modern maze of utilities.

(3) The location of underground utilities by digging or probing should be undertaken only as a last resort, and then only with the approval and supervision of the company involved. Some techniques that work are the use of a magnetic locator, a dip needle, or even “witching” for pipes or lines underground.

4-4. As-Built Surveys

a. Definition: As-built surveys are surveys compiled to show actual condition of completed projects as they exist for record purposes and/or payment. Since many field changes occur during construction, both authorized and clandestine, surveys are regularly completed to check the project against plans and specifications.

b. Requirements. As-built surveys are usually a modified version of the preliminary survey that was originally required to plan the project. This is particularly true of road, railroad, or watercourse relocation projects. These projects are all of the route survey nature. The as-built survey, out of necessity, is this type also. Typical items checked are:

- Alignment
- Profile or grade
- Location of drainage structures
- Correct dimensions of structures
- Orientation of features
- Earthwork quantities, occasionally

c. Methodology. For route-survey-type projects a traverse is usually run and major features of the curve alignment are checked. Profiles may be run with particular attention paid to sags on paved roads or other areas where exact grades are critical. Major features of road projects that are often changed in the field and will require close attention are drainage structures. It is not uncommon for quickly changing streams to require modification of culvert design. Therefore, culvert and pipe checks are critical. Typical items that should be checked for all major drainage structures are:

- Size (culverts may not be square).
- Skew angle (several systems in use).
- Type or nomenclature (possibly changed from plans).
- Flow line elevations are very important and should be accurately checked.
- Station location of structure center line with regard to traverse line should be carefully noted.

(1) Utilities that have been relocated should be carefully checked for compliance with plans and specifications. Incorrect identification of various pipes, tiles, and tubes can result in difficulties. Since the subject is somewhat complicated it is important to keep track of this type of information for the future.

(2) Project monumentation is sometimes a requirement of as-built surveys since it is common to monument traverse lines and baselines for major projects. Their location should be checked for accuracy. In many areas it is common for such monumentation to be done by

maintenance people who are not at all familiar with surveying and therefore the work is not always as accurate as would be desired.

Section I. Plane-Table Surveys

4-5. General

The Egyptians are said to have been the first to use a plane table to make large-scale accurate survey maps to represent natural features and man-made structures. Because of our familiarity with maps it is easy for us to stand on a site with a scaled map, orient it in the direction of the features on the ground, and accurately picture ourselves on the map and on the ground itself. The survey map is the basic starting element of a civil engineering project. Plane table surveying has, for most purposes, been replaced by aerial photogrammetry and total stations, but the map is still similar. The plane table is well suited for irregular topography existing in natural landscapes. Man-made symmetric landscapes such as cities and military installations are more suited to total station or transit-tape methods. One advantage the plane table has over the transit-tape point locations is the accuracy of contour lines drawn by an experienced surveyor. The surveyor will interpolate enough controlling points to find the contours. Parts of contours are artistically drawn to capture the actual landscape without having to locate the focus of points describing a trace contour. The topographic map is finished in the field by plane table method. Digitizing plane table sheets from experienced topographers provides more accurate contours than contours interpolated from point location. The plane table method can also be used to spot-check existing maps with small scales (less than 1" = 100'). Allow for paper shrinkage due to ambient outdoor weather conditions. Plot digital maps on fresh mylar to avoid this problem.

4-6. Plane-Table Topography

A topographic survey is made to determine the shape or relief of part of the earth's surface and the location of natural and artificial objects thereon. The results of such a survey are shown on a topographic map. A plane table, telescopic alidade, and stadia rod are used to locate the required points, and they are plotted on the plane table sheet as the measurements are made. The use of the plane table method in obtaining topography affords the following advantages:

a. The terrain being in view of the topographer reduces the possibility of missing important detail.

b. Detail can be sketched in its proper position on the plane table sheet from a minimum of measurements. This applies especially to the representation of relief where the contour lines can be sketched between plotted points.

c. A greater area can be accurately mapped in a given period of time.

d. Triangulation can be accomplished graphically to avoid office computation.

e. Office work is reduced to a minimum.

4-7. Plane-Table Triangulation

Plane-table triangulation consists of the location of many points on a plane-table sheet through the use of the so-called pure plane-table methods. These involve only the use of the plane table and alidade, and in theory the operations may be executed by one man. In practice one or more systems are employed and auxiliary methods such as stadia are used. The area that is to be mapped is outlined on the plane-table sheet by means of a projection on which the initial horizontal-control points have been plotted. Plane-table triangulation starts from these control points, and by means of a plane-table method of intersection and resection the necessary points on which to tie the topographic details of the map are "cut-in" or located on the plane-table sheet.

a. Equipment. The plane table consists essentially of a drawing board that is supported by a tripod and used in connection with an alidade. The board can be leveled and also turned in any horizontal position and can be clamped when properly set. When in use the plane table and its support must never move. Once set or oriented the greatest care must be taken that the position is not disturbed. The topographer must not lean on it or against it.

b. Setup. After leveling the plane table place the alidade on a line connecting the station occupied with one of the triangulation points farthest away. Revolve the table until the farthest signal is bisected by the vertical wire of the alidade and clamp the table. Verify the orientation by sights to additional visible triangulation stations. Now make the circuit of the horizon systematically and take foresights to prominent objects, such as, signals, towers, chimneys, flagpoles, monuments, church steeples, and definite points on schoolhouses, dwellings, barns, trees, or spurs. Draw the lines of sight with a chiseled edged 9H pencil along the square edge of the alidade, being careful always to hold the pencil at the same angle and to see that the contact of rule and paper is perfect. Get azimuths of long straight stretches of roads and railroads whenever possible. Stadia may well be used to locate road forks or objects in the immediate vicinity of the station. From time to time while making observations and on the completion of the work at each station, check the orientation of the plane table in order to see if there has been any movement.

c. Vertical angles. After all the sights have been taken adjust the level of the alidade and read vertical angles to the points whose elevations are desired. Angles that are read to the principal control points in this scheme should be checked.

d. Station elevation. The elevation of each plane-table station should be determined by means of vertical angles taken either to specifically located benchmarks or to other plane-table stations whose elevation has been previously determined. In general, the principal stations from which the greatest number of vertical angles are to be taken should be connected by means of reciprocal vertical angles taken under differing conditions. The final elevation of each station in the net should be determined by means of a weighted adjustment of the observed differences in elevation. In measuring important vertical angles, such as those to other stations or to points on level lines, all readings should be checked by reversing both level and telescope and by using different positions of the vertical arc. This can be accomplished by placing a plotting scale or some other flat object under one end of the alidade ruler.

e. Records and computations. Plane-table stations may be designated in the same manner as traverse stations. Such designations should be written on the plane-table sheet and also entered in a suitable notebook. A brief description of objects sighted may be noted on the plane-table sheet and written along the line of sight. Complete descriptions should be recorded in the notes. The notebook should also contain the vertical-angle records.

(1) Distances between stations and located points whose differences in elevation are to be computed will be scaled by means of a boxwood scale provided for the purpose and graduated for the field scale. Computation of differences in elevation is facilitated by the use of prepared tables. Care should be exercised in noting and in allowing for the height above ground for both the alidade and the point sighted. The proper corrections for the refraction should be applied whenever appropriate.

(2) When the work on the initial station is finished repeat the operation on the station at the other end of the base and on as many additional stations as may be necessary to complete the work. If practical, all triangulation stations that have been plotted on the sheet should be occupied. The point of intersection of the lines drawn to same objects determines its location, but all intersections should be verified by a sight from a third position. Additional stations may be made at intersected points, at points to each of which a single foresight has been drawn on the plane table sheet, and at points whose locations may be determined by the "three point method." Figure 4-1 illustrates this method.

f. Suggestions. A signal should be erected where necessary to mark the place of a station for future reference. Care should be taken to prolong on the plane-table sheet a line that may later be used for an orientation. In areas of great relief and difficult access every opportunity should be taken to contour topographic features. These include bottoms of canyons, rock exposures along canyon walls, and ground surfaces in heavily timbered areas.

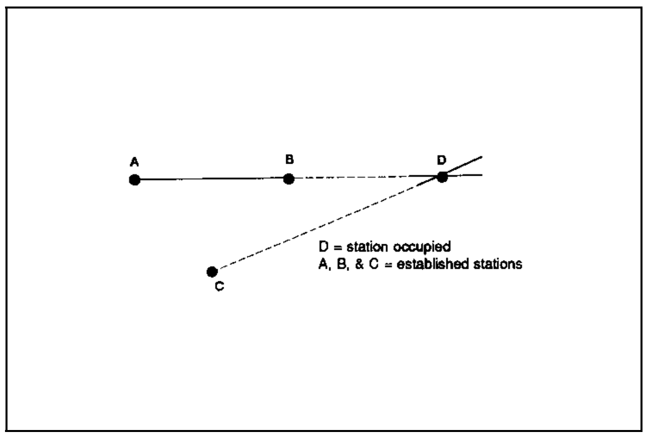

Figure 4-1. Resection - orientation by backsighting

4-8. Plane-Table Resection

a. The location of a plane-table station may be obtained by the method of resection (Figure 4-1), which is stronger than the three-point method. Location by resection involves two separate operations performed on two different stations, and in practice any length of time may lapse between the two operations. The resection method is of limited use since it involves a foresight from a previous station and the erection of a signal on the proposed station point or the positive identification afterward of a point or direction sighted. Foresight lines that are to be used for a location by resection should be drawn on the plane-table sheet exactly through the point representing the station from which the sights are taken and to the full length of the alidade ruler. The line through the point representing the occupied station should be light and care should be taken to hold the chiseled pencil point directly against the ruler. The foresight line need only be drawn through the approximate position of the new station and at the extreme end of the ruler. Doing so will avoid adding unnecessary lines to the sheet. After drawing the line look through the telescope again and test the plane-table orientation to ensure that the table has not moved. Similar foresight should be taken when considering perspective station points whose locations may be obtained at some future time by resection.

b. To locate a new station which is being occupied and to which a long foresight has been previously drawn, orient the plane-table sheet by placing the alidade ruler in the reverse direction of the line sighted. Swing the plane-table board until the station from which the foresight is drawn is seen behind the center wire and clamp the plane-table board. The station being occupied is on this foresight line. The location is determined by resection from other plane-table stations whose directions are most nearly at right angles to the foresight line that they are to intersect and whose distance is less than that of the station from which the foresight line was drawn. Center the ruler on the plotted position of one of these stations. Swing the telescope until the signal mark at that station is behind the center wire and draw a line against the ruler to intersect the foresight line. The intersection marks the location of the occupied station. The intersection should be checked by at least one other resection from another station. The two resected lines should cross the foresight line at the same point.

4-9. Plane-Table Two-Point Problem

In Figure 4-2, a and b are the plotted positions of visible inaccessible control stations A and B. It is desired to locate and orient the plane table at an unplotted station C. Use the following procedure:

a. Select an auxiliary point D where the resection lines from A and B will give a strong intersection (greater than 30°). D must be located with respect to C so that cuts on A and B from C and D will also give strong intersections.

b. Set up the plane table at point D and orient by eye or with a compass.

c. Resect on A and B. The intersection of these resection lines is d', the tentative position D.

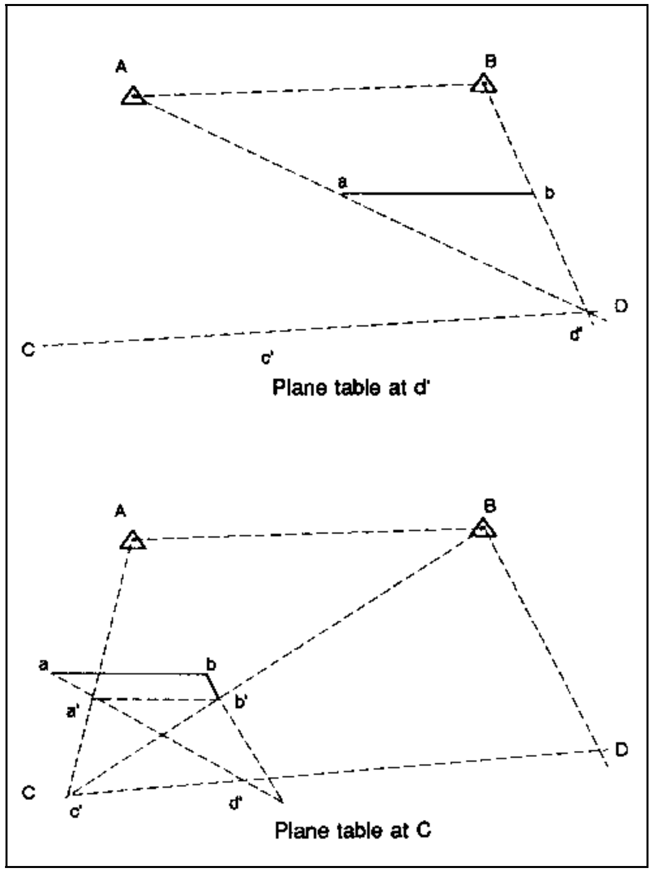

Figure 4-2. Two-point problem

d. Draw a ray from d' toward C. Plot the point c' on this ray at a distance from d' corresponding to the estimated distance from D to C.

e. Set up the plane table at point C and orient by a backsight on D. The error in this orientation is the same in magnitude and direction as it was at D.

f. Sight on A and draw the ray through c' intersecting the line ad' at a'. In a like manner, sight B to obtain b'.

g. Quadrilateral a'b'd'c' is similar to ABDC as the line a'b' will always be parallel to the line AB, the error in orientation will be indicated by the angle between line ab and a'b'. To correct the orientation, place the alidade on the line a'b' and sight on a distinctive distant point. Then place the alidade on the line ab and rotate the plane table to sight again on this point. The plane table is now oriented and resection on A and B through points a and b establishes the position of the desired point C.

4-10. Plane-Table Traverse

a. Traversing consists of much more than getting direction and distance, though these are absolutely essential features. All essential topographic features on each side of the line are to be obtained at the time the traverse is made.

b. Accuracy of the plane-table traverse depends on two factors: obtaining and plotting of distances and the orientation of the plane table.

c. Distances are obtained by stadia or tape, and the orientation is made by magnetic needle by backsight and foresight.

d. When the needle is used the accuracy of the orientation is dependent on the freedom from local attraction and the length of the needle. For these reasons it is necessary to avoid the use of the compass near railroads, electric transmission lines, or large bodies of steel or iron, and in volcanic regions. No plotted line should be greater than the length of the needle.

e. The method employed in determining distances will be governed by the character of the country and the scale of the work. Traverse lines should be run along roads, ridges or streams, or at intervals in timbered country when necessary. When the needle is used to set up alternate stations, intermediate stations should be used as turning points.

f. Streams near the roads should be mapped as accurately as the skill and experience of the topographer will permit.

g. When traversing railroads, frequent locations by the three-point method should be made whenever possible. The line should be extended by means of foresights and backsights. If this is not practical and it becomes necessary to rely on a needle, it is important to set up the plane table a sufficient distance from the rails to prevent their influence on the needle. The distances can be obtained advantageously by measuring a rail and counting the number of rails between stations.

h. To properly plot a new station place the fractional scale division on the old point. Then prick the new station with a needle at the even division at the end of scale. This operation should be performed with great care since more closure errors are attributed to careless plotting than to any other cause.

4-11. Plane-Table Stadia Traverse

When using plane table and stadia traverse, instrumental measurement of the distances in elevation gives sufficient control to permit considerable sketching to be done on either side of the line. Wherever possible, as in regions of low relief, elevations should be determined by using the alidade as a level and a rod as a level rod.

4-12. Plane-Table Three-Point Orientation

The three-point method involves orienting the plane table and plotting a station when three stations (established control) can be seen but not conveniently occupied. The following procedures should be used:

a. Set up the plane table at the unknown point P (Figure 4-3) and orient by eye or compass.

b. Draw rays to the known points A, B, and C. These rays intersect at three points (ab, bc, and ac) forming a triangle known as the *triangle of error*. The point ab denotes the intersect of the ray to A with the ray to B. Points bc and ac are similar in their notation. If the plane table is oriented properly, the rays to the three known points will intersect at a single point (P) rather than three points found in *a* above. To accomplish this result, turn the plane table several degrees in azimuth, then construct a second triangle of error (a'b', b'c' and a'c').

c. Construct a third triangle of error (a"b", b"c", a"c"). A circular arc through points ab, a"b", and a'b' will pass through the point P; a circular arc through bc, b"c", and b'c' will pass through point P. These arcs should be sketched and the point P plotted.

d. Orient the plane table by sighting from the plotted position of P, to the plotted position of any of the three known points.

e. To check the solution, a fourth known station is observed when possible after completion of the three-point problem. If the ray through this fourth station does not intersect the other three rays at P an error has been made.

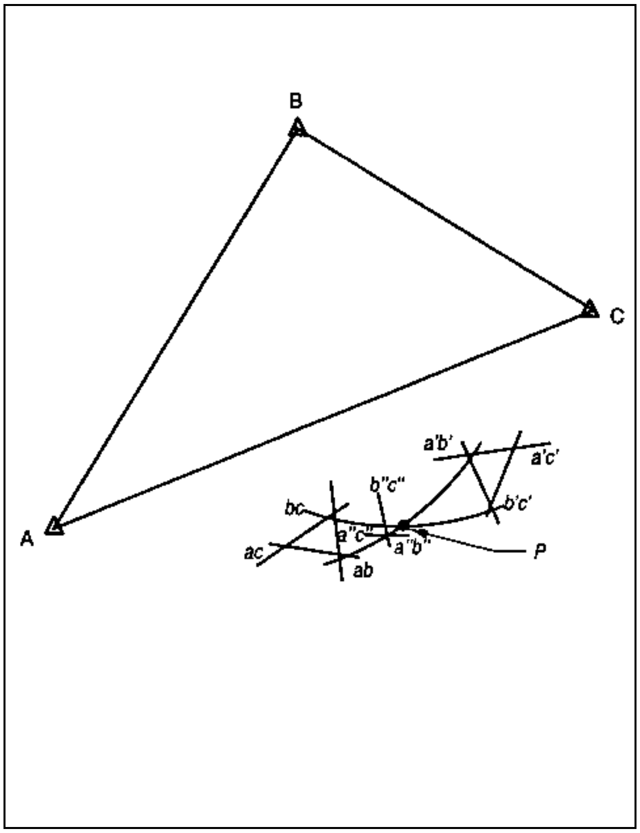

Figure 4-3. Three-point problem

4-13. Contouring Methods

Contours may be mapped from plane table setups that are made directly over or adjacent to the country that is to be mapped. These areas include a traversed road, a table that is kept stationary while it is circled by one or more moving rods, or from a plane-table station that overlooks the distant country to be contoured. These three methods of contouring are described below:

a. Contouring from a traversed line. In regions where the principal control is obtained by different kinds of traverse the usual procedure is to first plot the contour crossings and other contours on or near the traversed road or other traverse lines. Then extend the contouring out on both sides of the traverse line as far as good visibility and locally established control warrants. When the visibility from a traverse line is poor, points off the line may be used. Contouring from a traverse line may also be advantageously supplemented by having the rod held occasionally at salient points in the topography opposite the line. When all signs of a road circuit or other large traverse circuits have been traversed and mapped, traverses should be extended into or across the unmapped interiors giving sufficient traverse control to enable the topographer to complete the mapping. Such interior traverses may be run across open country or along ridges or streams. The topographer may occupy favorable interior viewpoints with the plane table and reset from a previous traverse location in order to obtain a plane table location.

b. Contouring in open country. In open country of low relief where little contouring can be done from single plane-table setups, one or two rodmen and a recorder can be advantageously used. When two rodmen are employed, each holds a rod on different sides of the plane table and at salient points in the topography. Each rodman advances in the direction of the proposed mapping. As soon as the sites become too long or are about to be obscured both rodmen should hold their stations, and these points should be used as tuning points in the line. The mean of the two readings should be used in determining the elevation of the new plane-table station that is made beyond the points held by the two rodmen. The rodmen then advance as before. Plane-table locations may be obtained as in contouring from a traverse line above or from stations below. To use this method effectively the topographer should employ signals between the table and the rod and between rods. The topographer should fully instruct the rodmen in their duties since much depends upon their activity and resourcefulness. The readings that result from successive rod sites may be plotted as they are taken, or depending on local conditions they may be allowed to accumulate and plotted after the series has been completed.

c. Contouring from station. In open country of bold relief, where all the features are plainly visible, contours can best be delineated from plane-table stations overlooking the country to be contoured without the running of a traverse line or the use of a rod. The method of contouring from plane-table stations involves the use of plane-table triangulation and the three-point method. Woodland country as well as open country may be contoured from stations, provided a sufficient number of outlooks can be found from which a satisfactory view and a good determination of positions can be obtained. In the construction of contours from a station the location and use of drainage lines is important.

(1) Contouring from a station is dependent on supplemental control that is obtained by the location on the plane-table sheet by intersection method of many of the salient points on the surface that is to be contoured. For this reason little sketching can be done from the first station other than form lines which are later converted into placed contour lines. In planning the order of

plane-table stations careful consideration must be given to the need for the siting of many points ahead. A sufficient number of such sites may be intersected from subsequent stations and used as a basis for contour construction. Vertical angles may be taken from the points when they are first sited, after the points have been intersected, or at both times. In either procedure the elevations must be computed and the contours placed on the map as soon as intersections are obtained.

(2) The elevation of the plane-table stations must be determined from a carefully executed series of reciprocal vertical angles taken between the principal stations in the quadrangle. At least one station must be directly connected with a level benchmark by means of reciprocal vertical angles measured under different conditions.

d. Contour skeleton outline. Before mapping the contours that are to represent the distance relief feature a skeleton outline of that feature should be prepared. The quality with which this is accomplished depends upon the accuracy, speed, and ease that the contours are placed on the map. Lacking an outline an experienced topographer will make a suitable skeleton outline of the drainage and ridge lines before attempting the construction of the contour lines.

(1) The landscape that is to be contoured should be divided into its separate features or unit masses; such as mountains, hills, and spurs. After sufficient control has been established through intersection methods, each feature segregated should have its natural drainage lines and boundaries sketched in. This should include tangents drawn to the salient points as well as located points being used as control for the placing of the drainage lines. Similarly, ridge and crest lines may be outlined. It is best to use convex forms as unit masses, such as spurs and lateral ridges. The intermediate drainage lines should be used as boundaries.

(2) In determining elevations based on vertical angles remember that large angles must be supplemented with accurate measurements of distance. Small angles based on measurement of distances that are approximate will only yield useful elevations for contour work.

(3) Each separate unit mass should be as completely contoured as control and visibility permits before the contouring of another feature is begun. Should control alone be lacking, form lines should be lightly sketched in and advantage taken of a favorable view point. An effort should be made to cut-in the lacking control. By treating each mass as a separate unit, each can be delineated with its own characteristic shape.

4-14. Locating and Plotting Detail

Detail points are plotted to scale on the plane-table sheet with respect to the plotted position of the occupied station. Detail points are normally located and plotted by the radiation method and instances are measured by stadia. Rays and instances are plotted directly on the plane-table sheet. The ground elevation at each detail point is determined and the elevation is shown on the plane-table sheet beside the plotted position for the point.

a. Features shown on the map for which detail points must be located include the following:

(1) Works of man, such as buildings, roads, ridges, dams, and canals.

(2) Natural features, such as streams, lakes, edges of wooded areas, and isolated trees.

(3) Relief.

b. On large-scale maps it is often possible to represent the true shape of features to scale. On small-scale maps buildings and other features must often be symbolized with the symbols centered on the true position but drawn larger than the scale of the map. Such detail is portrayed on the map by means of standardized topographic symbols.

c. Detail points and elevations for contouring are usually located at key points of distinct changes in ground slope or in the direction of a contour. Such key points are located at the following positions:

(1) Hill or mountain tops.

(2) On ridge lines.

(3) Along the top and foot of steep slopes.

(4) In valleys and along streams.

(5) In saddles between hills.

d. Contour lines are drawn on the map by logical contouring. Ground elevations are determined at key points of the terrain, and these positions are plotted on the plane-table sheet. After a number of key points have been located (usually from an occupied station) and plotted, sketching of the contour lines is started.

e. Figure 4-4 shows a portion of a plane-table sheet on which some contour lines have been drawn. Key points are shown in this figure with the interpolated positions of the contour lines plotted. Contours should be drawn as the plane tabling progresses so that the topographer can utilize the view of the terrain when drawing the contours.

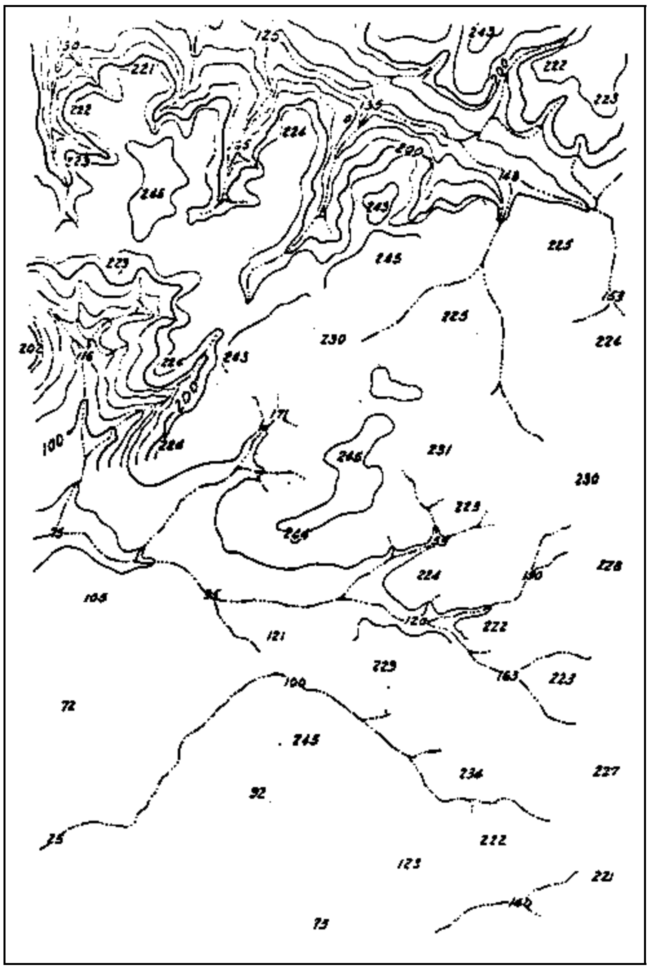

Figure 4-4. Contouring

f. When contouring, it must be remembered that stream and ridge lines have a primary influence on the direction of the contour lines. Figure 4-5 shows several typical forms often encountered in contouring. It should be noted that the contour lines crossing a stream follow the general direction of the stream on both sides and cross the stream in a fairly sharp “V” that points upstream. Also, contour lines curve around the nose of ridges and cross ridge lines at approximate right angles.

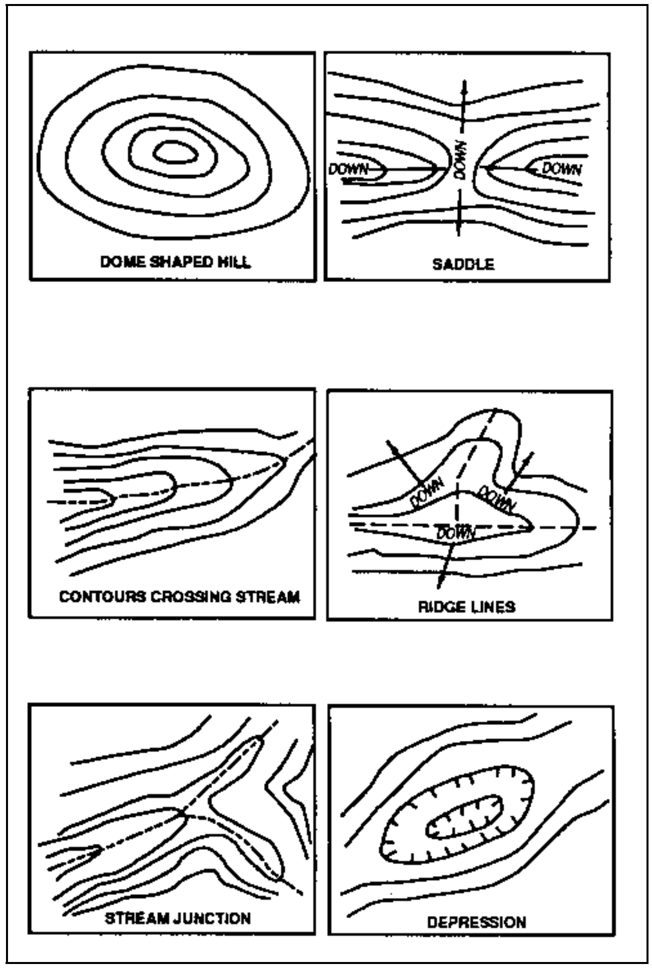

Figure 4-5. Typical topographic forms

g. Every fifth contour line should be drawn heavier than the others, and the elevations of these heavier lines should be shown at frequent intervals. These heavy, numbered contour lines are those representing multiples of the 5-, 10-, 25-, 50-, or 100-foot elevations. For example: with a 2-foot contour interval, the 10-foot contour lines would be drawn heavier and numbered.

h. Practical hints for plane table mapping are:

(1) The topographer should face the area he is contouring.

(2) Contour lines should not be drawn beyond the determined and plotted elevations.

(3) The pencil should be kept sharp at all time.

(4) Accurate plotting of distances is critical. For this reason, plane-table traverses should be kept to a minimum.

(5) When drawing a ray toward a detail point draw only a short line near the plotted position for the detail point.

(6) The magnetic needle can often be used to orient the plane table approximately at the start of resection problem.

(7) The topographer should not remain at the plane table, but should move around enough to acquaint himself thoroughly with the terrain.

4-15. Plane-Table Equipment Checklist

The equipment listed below should be on hand.

- Alidade
- Plane-table drawing board
- Plane-table tripod
- Thumb tacks (to secure map or plane-table sheet)
- Sewing needles
- Polymer 6J - 9H pencil leads for mylar
- 6H - 9H pencil leads for vellum
- 3H - 9H pencil leads for paper
- Plumb bob
- Cloth tape or reel chain
- Pounce (powder to keep black marks off the map and help the straightedge to glide)
- 25-foot topo rod
- Plane-table sheets

4-16. Plane-Table Setup Hints

a. Rotate the plotted point on the plane table over the point on the ground and in the direction of the backsight before completing the setup. Do this before placing the alidade on the table. The reason is the plotted point is generally not in the center of the plane-table board. Thus, any rotation about the Johnson head will send the plotted point away from the point on the ground. The wing nut for rotation is the lower nut or the one located furthest below the plane table.

b. Secure (snug, but not too tight) the Johnson head-ball and socket wing nut for leveling (closest to the plane table). Make sure the plotted control point has been plumbed over the actual control point. Place the alidade in the center of the plane table or over the Johnson head. With one hand holding the plane table fixed, carefully loosen the leveling wing nut and level the table with the bull's eye bubble. Feel with the free hand and secure the upper wing nut (leveling nut). Check the setup for level and use a plumb bob to ensure the plotted point is over the actual point.

4-17. Plane-Table Notekeeping

The following items must be recorded at every setup:

a. Measure up from control point on the ground to the alidade trunion axis (height of instrument (h.i.) above ground). The alidade must be positioned over the point with the straightedge against the needle.

b. No setup will exceed 0.1 foot in distance between the plotted control point plumbed over the actual control point when the station is being occupied as a setup.

c. After setup initialization (table oriented and table level) a horizontal check on at least two points other than the point being occupied is necessary. One of these two points may be the backsight. Distance as well as alignment must be recorded to the accuracy (acceptable tolerance) of the survey in progress. If no other checkpoint is visible, a traverse must establish mapping control. This traverse can be performed with stadia using the plane table to mark the points. Two points other than the point occupied will also be used to check the vertical. A beamen arc will be used to check one of these elevations. Record all observations for checks neatly into the fieldbook.

d. The distance between plotted points on the plane-table sheet shall not exceed 1 inch. For example, a plane-table survey to be performed at 1" = 20' shall have ground shots spaced no greater than 20-foot spacing.

e. No point plotted on the plane-table sheet shall have a vertical error greater than 0.3 foot.

f. All man-made concrete structures will be located vertically to the hundredth of a foot.

g. Hand-drawn contours shall be established by interpolation in two directions for every tick made on the plane-table sheet.

4-18. Plane-Table Location Details

The following details should be noted on the map:

a. Railroads. Location of one track, track widths, number of tracks for parallel tracks.

b. Bridges. Bridge type, center lines, type of surface, number of lanes, bridge piers, fender systems, distance from center line to bottom of typical bridge beam, utility cables.

c. Tunnels. Center lines, approach islands, utility cables, develop a typical tube section.

d. Airports. Center-line runways and magnetic azimuth of runway, runway thresholds, runway lights (amber), taxiways, taxiway lights (blue), possible obstructions to aircraft (tall trees, towers, buildings), control tower plus height, navigation aids (ILS localizer, VASI, VOR, VORTAC, NDB, LOM, RVR). NOTE: Control monuments are usually located in the area of the runway threshold.

e. Dams. Dam type, center line, typical cross-section, toe of slope, water line.

f. Canals. Bulkheads, piers, pilings, dolphins, cable crossings, riprap or armor stone perimeters.

g. Roads. Number of lanes, type of surface, center lines, all edges of pavements, drainage ditches, driveway entrances, driveway culverts, road intersections, curb and gutter (vertically to 0.01 foot).

h. Utilities. For underground utilities call the proper agency for line location stakeout. Locate all stakes set in the field by this agency. Failure to locate all stakes could result in serious construction injuries.

- Power poles with pole number and agency names
- Overhead powerlines
- Overhead telephone lines
- Overhead cable television lines
- Storm sewers (curb inlets, yard inlets, etc.) storm sewer throat size, pipe sizes, inverts in and out
- Sanitary sewers - low side rim elevations and all inverts
- Gas lines - note diameters - tees
- Water lines - note diameters - tees
- Fire hydrants - never place a benchmark on the top nut, use a bell bolt
- Underground electric - locate transformers, building connections
- Underground gas petroleum - locate all tees and cutoffs

i. Buildings. Schools, churches, homes, industrial, office, barns, warehouses.

- Locate at least three corners and chain all distances completely around the perimeter of the structure. Include any overhangs (OH)
- Show overhead, ground surface, and underground utilities serving the building
- Locate any ruins of buildings
- Tanks - water, gas, oil, etc
- Wells
- Pumps
- Septic fields if possible
- Fences
- Walls
- Sheds, note type of foundation

j. Cemetery. Carefully locate all graves.

k. Waterways.

- Bulkheads
- Piers
- Dolphins
- Piles
- Docks
- Slips - concrete or wood
- Buoys
- Oyster grounds
- Dikes
- Riprap -- Cobble size to armor stone size

l. Natural features.

- Ridge lines
- Center-line swales
- Change in slope
- Valley lines
- Creeks (top of banks and center line)
- Springs
- Ponds
- Lakes
- Edge of woods
- Swamps
- Rock outcroppings
- Mines
- Trash dumps

Section II. Electronic Total Station Surveys

4-19. Electronic Total Stations

Traditionally, surveying has used analog methods of recording data. The present trend is to introduce digital surveying equipment into the field. The fastest digital data collection methods are now done by electronic total stations. Total stations have dramatically increased the amount of topographic data that can be collected during a day. Figure 4-6 is a flow diagram of the digital information from field to finish. The method is well suited for topographic surveys in urban landscapes and busy construction sites. Modern total stations are also programmed for construction stakeout and highway center-line surveys. Total stations have made trigonometric levels as accurate as many of the differential level techniques in areas possessing large relief landforms. These instruments can quickly transfer 3D coordinates and are capable of storing unique mapping feature codes and other parameters which in the past could only be recorded on paper media such as field books. One of the best features of the total station is the ability to electronically download directly into a computer without human blunders. The use of electronic storage can result in a blunder of the worst magnitude, if reflector prism HI's are changed and not noted. Feature codes entered in error are also potential map errors which can be detrimental to an entire project.

a. Total station surveys are like plane-table surveys. The advent of the total survey station has made it possible to accurately gather enormous amounts of survey measurements quickly. Even though total stations have been around for more than 20 years, they are just now beginning to become popular among the general surveying and engineering community. In the last 10 years total stations and data collectors have become common field equipment. As time progresses they will increase in number and variety. The majority of survey firms are using data collectors today.

b. Reasons for implementation. In the early 1980's the surveying instrument manufacturers introduced what has become a true total station, redefining the term by creating an entirely electronic instrument. In other words, the readout on the display panels and the readout from the EDM are in a digital form. This feature eliminated the reading errors which can occur with an optical theodolite. Also with the advent of the electronic theodolite came the electronic data collector, thus minimizing both the reading errors and the writing errors. Since the information in the data collector is interfaced directly to a computer, errors which occur in transferring the field information from the field book to the computer are eliminated. At this point one can measure a distance to a suitable range with an accuracy of better than 5 millimeters plus 1 part per million, and angles can be turned with the accuracy of one-half arc second, all accomplished electronically. The vast increase in productivity which is available to us from what we had previous to 1980 is due to this modern equipment. In most land surveying situations, the normal crew size can be reduced to two when equipped with an electronic theodolite. Since the data acquisition time is so fast in some situations three men are warranted when it is possible to utilize two prism poles. This often results in an overall reduction in man-hours spent on the job.

c. Field-finish surveying capabilities. Figure 4-6 depicts the capability of an electronic total station to perform direct field-finish mapping products.

4-20. Field Equipment

a. Modern electronic survey equipment requires surveyors to be more maintenance conscious than they were in the past. They have to worry about power sources, downloading data, and the integrity of data, including whether or not the instruments and accessories are accurately adjusted and in good repair. When setting up a crew to work with a total station and data collector, it is helpful to supply the party chief with a checklist to help the crew maintain its assigned equipment and handle the collected data upon returning to the office. It is also important that each crew be supplied with all necessary equipment and supplies. These should be stored in an organized and easily accessible manner.

b. Let us first examine an equipment list that will assure the survey crew (two-person crew, consisting of a party chief/rodperson and a notekeeper/instrument person) a sufficient equipment inventory to meet the general needs of boundary, layout, and topographic surveys. Also, assume that this crew has a regular complement of supplies such as hammers, shovels, ribbon, stakes, lath, and safety equipment. This discussion will be confined to what is needed to maximize productivity when using a total station with data collector.

c. The minimum equipment inventory required is as follows:

- Total station
- Data collector
- Batteries for 14 hours of continuous operation
- 3 tripods
- 3 tribrachs
- 3 target carriers
- 1 plumbing pole
- 4 target holders
- 4 reflectors
- 4 target plates

d. With this equipment inventory, a two-person field crew will be able to handle most survey tasks that are routinely encountered in day-to-day operations. An additional tripod, plumbing pole, carrier, tribrach, and reflector would give the crew even greater flexibility, and allow them to handle many projects more efficiently. It is also helpful for the field crew to have a convenient place to store their assigned equipment. Equip crews with briefcase-sized cases that will hold three tribrachs, four reflectors with holders, three carriers, and four target plates. A hard camera case or pistol case works well for this purpose. With all the components stored in one place, it makes inventory of the equipment easy and reduces the chance of equipment being left at the job site. This also allows for proper equipment maintenance.

4-21. Equipment Maintenance

The following checklist will aid each crew in properly maintaining and keeping inventory of their assigned equipment. At the end of each workday the party chief should check that the following duties have been performed:

- Clean all reflectors and holders. A cotton swab dipped in alcohol should be used on the glass surfaces. A crew member can do this during the trip back to the office.
- Clean tribachs. They should be dusted daily.

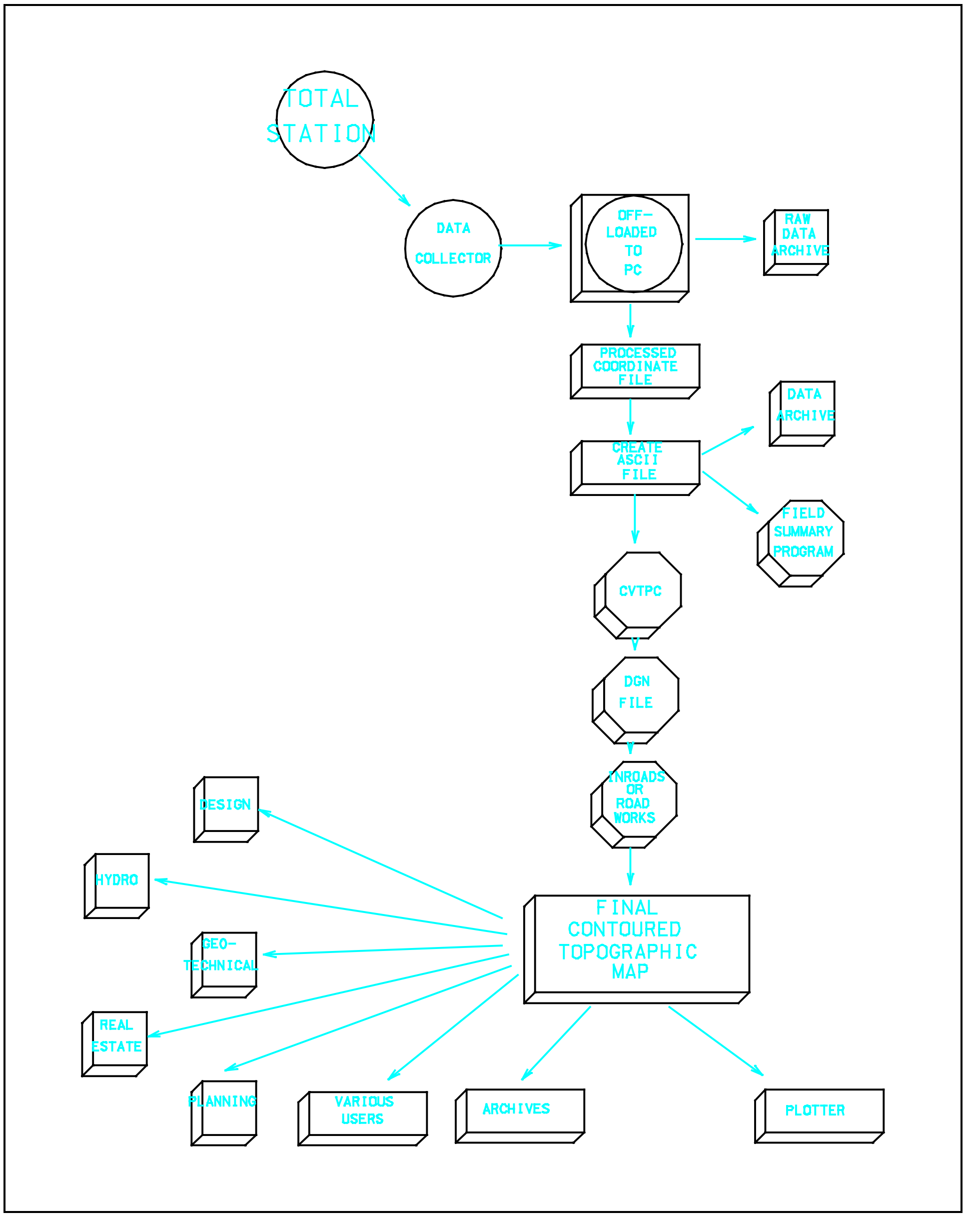

Figure 4-6. Total station data - field to finish

- Remove dust from all instruments. A soft paint-brush or a shaving brush works well. If an instrument has been exposed to moisture, thoroughly dry it and store in an open case.

- Download the data collector to the computer.

- Backup all files generated from the download and check the integrity of the backup files before erasing the field data from the data collector.

- Clean batteries and connect to charger. Some batteries require a 14- to 20-hour charge, so one set of batteries may have to be charged while a second set is in operation.

4-22. Maintaining Battery Power

One of the biggest problems faced by the users of total stations with data collectors is maintaining an adequate power supply. There are several factors that should be considered when assessing power needs:

a. Type of survey. A topographic survey entails much more data than a boundary survey. Normal production in a topographic mode is 200 to 275 measurements per day (350 shots per day is not uncommon). A boundary survey can entail making 16 measurements or so from each traverse point and occupying 10 to 15 points per day. Determine the number of measurements that you would normally make in a day and consult the manufacturer's specifications to determine the number of shots you can expect from a fully charged new battery.

b. Age of batteries. Keep in mind that batteries will degrade over a period of time. This means that a new battery, with sufficient power for 500 measurements when new, may only be capable of 300 measurements after a year of use.

c. Time needed to charge batteries. Some batteries take up to 14 hours to fully charge. If the work schedule will not permit 14 hours for charging a second set of batteries, a battery with adequate power to supply the instrument for more than 1 day should be purchased.

d. Power requirements of equipment. Older recording total stations that write data to tape will use up to three 7-amp hour batteries per day. A newer instrument by the same manufacturer will take 3 days to use one battery. Since newer instruments use far less power than those on the market 6 years ago, this should be considered in determining power needs.

(1) In addition to the proper assessment of power need, a record of the history and current status of the power supply should be readily available. When batteries begin to get weak there is generally a rapid deterioration in their performance. To monitor the performance of a particular battery record the serial number in a battery log book. If problems arise with a particular unit, check the log to see when the battery was purchased or when it was last recelled. Next try discharging and recharging the battery. If performance is still not up to speed, have it checked to determine the weak cell and replace it. If the battery was not new or was recelled in the last year, recell the entire unit. When one cell goes the next one is usually only a charge or two from failure. The cost of having a battery recelled is minimal when considering the cost of lost worktime due to power failure.

(2) Also record the date the battery was charged on the shipping label that is attached to the battery box. When the battery is fully used simply cross out the date, thus eliminating the confusion of not knowing which battery needs to be charged. Monitor the shelftime of the battery, and if it exceeds 10 days, recharge it. This keeps the power supply at peak performance. Always consult the operator's manual for recharge specifications.

(3) It is always a good idea to have backup power available for that last 15 minutes of work. Most manufacturers can provide cabling for backup to an automobile battery. Some can even supply a quick charge system that plugs into the cigarette lighter.

(4) Power pointers are

- Assess power needs for the particular job
- Assess power usage of the equipment
- Monitor performance of each battery
- Monitor battery age, usage, and recell information
- Have one day's worth of backup power readily available

4-23. Total Station Job Planning and Estimating

An often-asked question when using a total station with a data collector is "How do I estimate a project?" To answer this question first examine the productivity standards expected of field crews.

a. Most crews will make and record 200 to 275 measurements per day. This includes any notes that must be put into the system to define what was measured. When creating productivity standards keep in mind that a learning curve is involved. Usually it takes a crew four to five projects to become familiar with the equipment and the coding system to start reaching the potential productivity of the system.

b. A two-person crew is most efficient when the typical spacing of the measurements is less than 50 feet. When working within this distance the average rod person can acquire the next target during the time it takes the instrument operator to complete the measurement and input the codes to the data collector. The instrument operator usually spends 20 seconds (+ or -) sighting a target, recording a measurement, and another 5-10 seconds coding the measurement.

c. When the general spacing of the data exceeds 50 feet, having a second rod person will significantly increase productivity. A second rod person allows the crew to have a target available for measurement while the first rod person is moving. If the distance of the move is 50 feet or greater, the instrument will be idle with only one rod person.

d. When dealing with strip topographic situations, data must be acquired every 3 feet along the length of the job. In urban areas the data may seem to be more dense, but the rights-of-way are generally wider. The rule of thumb of one measurement for every 3 feet of linear topography works very well for estimating purposes. Using this estimate the typical field crew will make and record between 350-500 measurements or 1,000-1,500 feet of strip topography per day. Typically, a two-person crew equipped with recording total station and data collector pick up 1,250 feet a day. Depending on the office/field reduction software being used these data can produce both the planimetric and contour maps as well as transfer the data to an engineering design package with very little additional manipulation.

e. To estimate strip topographic production remember the following tips:

(1) Estimate one measurement for every 3 feet of project.

(2) If shots are greater than 50 feet, a second rod person adds to the efficiency of the crew.

(3) Expect a two-person crew equipped with a recording total station and data collector to pick up 1,250 feet per day.

f. Another question asked is “Who runs the rod?” Conventional location or topographic surveying often requires a three-person field party. The party chief is working at the instrument, recording the measurements and other information in the field book. The party chief is also responsible for gathering data and must pay close attention to the movements of the rodman. When the field crew consists of two people, it is often the party chief who runs the rod. When using the power of field-to-finish data collection, the experience and judgment of the rodman is an important factor. Most organizations have the party chief or senior field technician run the rod and allow the less-experienced person to operate the instrument. The rodman communicates codes and other instructions to the instrument operator who enters them into the instrument or data collector and takes the measurements. Given the ease of operation of total stations, that is a fast and easy way to train an instrument operator. This frees more experienced personnel to control the pace of the job and to concentrate on gathering the correct data.

g. Multiple rodmen make for increased productivity. Data collection provides a tremendous increase in the speed of performing field work by eliminating the need to read and record measurements and other information. Many organizations have reduced the size of their field crews by eliminating the notetaker. However, this person can be very useful as a second rodman.

(1) On jobs where a large number of shots are needed, the use of two (or more) rodmen has resulted in excellent time and cost savings. The rodmen can work independently in taking ground shots or single features. They can work together by leapfrogging along planimetric or topographical feature lines. When more than one rodman is used, crew members should switch jobs throughout the day. This helps to eliminate fatigue in the person operating the instrument.

(2) As an extension of the concept discussed above it is a good idea to have an experienced person running one rod and directing the other rodmen. If possible, each rodman and the instrument operator should have a radio or other means of reliable communication.

(3) Electronic data collection (EDC) has proved to be an extremely cost-effective means of gathering data. When competing against a grid or baseline and offset style of surveying, EDC often results in field time savings of over 50 percent. However, deriving a horizontal and vertical position on the located points is only part of the process. The ultimate goal is usually a map, showing either planimetrics, or contours, or both.

h. Total station productivity compared with other methods is shown in Figure 4-7, which depicts the enhanced productivity of a total station relative to traditional plane-table or transit-stadia methods. Time savings in design/construction layout is shown in Table 4-1.

4-24. Electronic Theodolite Error Sources

All theodolites measure angles with some degree of imperfection. These imperfections result from the fact that no mechanical device can be manufactured with zero error. In the past very specific measuring techniques were taught and employed by surveyors to compensate for minor mechanical imperfections in theodolites. With the advent of electronics the mechanical errors still exist but are related to in a different way. One must now do more

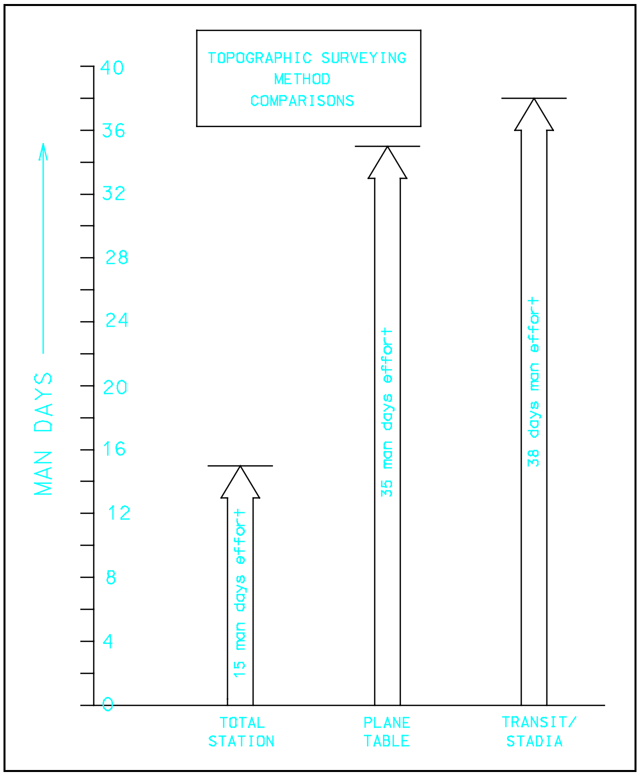

Figure 4-7. Topographic surveying method comparisons

Table 4-1
Total Stations Estimated Field Time Saved Over Other Methods

Application on Project Site	A % Time in Field Saved
Control traverse	5%
Topo	25%
X-Select	20%
Design layout	30%
As-built	35%

than memorize techniques that compensate for errors. One must clearly understand the concepts behind the techniques and the adjustments for errors that electronic theodolites now make. The following paragraphs provide the major sources of error when using a theodolite and also the particular method employed to compensate for that error.

a. Circle eccentricity. Circle eccentricity exists when the theoretical center of the mechanical axis of the theodolite does not coincide exactly with the center of the measuring circle. The amount of error corresponds to the degree of eccentricity and the part of the circle being read. When represented graphically circle eccentricity appears as a sine wave. Circle eccentricity in the horizontal circle can always be compensated for by measuring in both faces (opposite sides of the circle) and using the mean as a result. Vertical circle eccentricity cannot be compensated for in this manner since the circle moves with the telescope. More sophisticated techniques are required.

(1) Some theodolites are individually tested to determine the sine curve for the circle error in that particular instrument. Then a correction factor is stored in ROM that adds or subtracts from each angle reading so that a corrected measurement is displayed.

(2) Other instruments employ an angle-measuring system consisting of rotating glass circles that make a complete revolution for every angle measurement. These are scanned by fixed and moving light sensors. The glass circles are divided into equally spaced intervals which are diametrically scanned by the sensors. The amount of time it takes to input a reading into the processor is equal to one interval, thus only every alternate graduation is scanned. As a result, measurements are made and averaged for each circle measurement. This eliminates scale graduation and circle eccentricity error.

b. Horizontal collimation error. Horizontal collimation error exists when the optical axis of the theodolite is not exactly perpendicular to the telescope axis. To test for horizontal collimation error, point to a target in face one then point back to the same target in face two; the difference in horizontal circle readings should be 180 degrees. Horizontal collimation error can always be corrected for by meaning the face one and face two pointings of the instrument.

(1) Most electronic theodolites have a method to provide a field adjustment for horizontal collimation error. Again, the manual for each instrument provides detailed instruction on the use of this correction.

(2) In some instruments, the correction stored for horizontal collimation error can affect only measurements on one side of the circle at a time. Therefore when the telescope is passed through zenith (the other side of the circle is being read), the horizontal circle reading will change by twice the collimation error. These instruments are functioning exactly as designed when this happens.

(3) When prolonging a line with an electronic theodolite, the instrument operator should either turn a 180-degree angle or plunge the telescope and turn the

horizontal tangent so that the horizontal circle reading is the same as it was before plunging the telescope.

c. Height of standards error. In order for the telescope to plunge through a truly vertical plane the telescope axis must be perpendicular to the standing axis. As stated before there is no such thing as perfection in the physical world. All theodolites have a certain degree of error caused by imperfect positioning of the telescope axis. Generally, determination of this error should be accomplished by a qualified technician because horizontal collimation and height of standards errors interrelate and can magnify or offset one another. Horizontal collimation error is usually eliminated before checking for height of standards. Height of standards error is checked by pointing to a scale the same zenith angle above a 90-degree zenith in face one and face two. The scales should read the same in face one as in face two.

d. Circle graduation error. In the past, circle graduation error was considered a major problem. For precise measurements surveyors advanced their circle on each successive set of angles so that circle graduation errors were "meaned out." Current technology eliminates the problem of graduation errors. This is accomplished by photo-etching the graduations onto the glass circles. Next make a very large precise master circle and photograph it. An emulsion is applied to the circle and a photo-reduced image of the master is projected onto the circle. The emulsion is removed and the glass circle has been etched with very precise graduations.

4-25. Total Survey System Error Sources and How to Avoid Them

In every survey there is an accuracy that must be attained. The first step in using field and office time most effectively is to determine the positional tolerance of the points to be located. After this has been accomplished all sources of errors can be determined and analyzed. Some sources of errors are: pointing errors, prism offsets, adjustment of prism pole, EDM alignment, collimation of the telescope, optical plummet adjustment, instrument/EDM offsets, curvature and refraction, atmospheric conditions, effects of direct sunlight, wind, frozen ground, and vibrations. The accuracy required for each survey should be carefully evaluated. Each of the following factors can accumulate and degrade the accuracy of measurements.

a. Pointing errors. Pointing errors are due to both human ability to point the instrument and environmental conditions limiting clear vision of the observed target. The best way to minimize pointing errors is to repeat the observation several times and use the average as the result.

b. Uneven heating of the instrument. Direct sunlight can heat one side of the instrument enough to cause small errors. For the highest accuracy, pick a shaded spot for the instrument.

c. Vibrations. Avoid instrument locations that vibrate. Vibrations can cause the compensator to be unstable.

d. Collimation errors. When sighting points a single time (e.g., direct position only) for elevations, check the instrument regularly for collimation errors.

e. Vertical angles and elevations. When using total stations to measure precise elevations, the adjustment of the electronic tilt sensor and the reticle of the telescope becomes very important. An easy way to check the adjustment of these components is to set a baseline. A line close to the office with a large difference in elevation will provide the best results. The baseline should be as long as the longest distance that will be measured to determine elevations with intermediate points at 100- to 200-foot intervals. Precise elevations of the points along the baseline should be measured by differential leveling. Set up the total station at one end of the baseline and measure the elevation of each point. Comparing the two sets of elevations provides a check on the accuracy and adjustment of the instrument. Accuracy requirements may dictate that more than one set of angles and distances be measured to each point. Some examples are distances over 600 feet, adverse weather conditions, and steep observations.

f. Atmospheric corrections. Instruments used to measure atmospheric temperature and pressure must be correctly calibrated.

g. Optical plummet errors. The optical plummet or tribrachs must be periodically checked for misalignment.

h. Adjustment of prism poles. When using prism poles, precautions should be taken to ensure accurate measurements. A common problem encountered when using prism poles is the adjustment of the leveling bubble. Bubbles can be examined by establishing a check station under a doorway in the office. First mark a point on the top of the doorway. Using a plumb bob, establish a point under the point on the doorway. If possible, use a center punch to make a dent or hole in both the upper and lower marks. The prism pole can now be placed into the check station and easily adjusted.

i. Recording errors. The two most common errors associated with field work are reading an angle incorrectly and/or entering incorrect information into the field book. Another common (and potentially disastrous) error is an incorrect rod height. Although electronic data collection has all but eliminated these errors, it is still possible for the surveyor to identify an object incorrectly, make a shot to the wrong spot, or input a bad target height or HI. For example, if the surveyor normally shoots a fire hydrant at the ground level, but for some reason shoots it on top of the operating nut, erroneous contours would result if the program recognized the fire hydrant as a ground shot and was not notified of this change in field procedure.

j. Angles. As a rule, a surveyor will turn a doubled angle for move-ahead, traverse points, property corners, or other objects that require greater accuracy. On the other hand, single angles are all that are required for topographic shots.

4-26. Controlling Errors

A set routine should be established for a survey crew to follow. Standard operating procedure should require that control be measured and noted immediately on the data collector and in the field book after the instrument has been set up and leveled. This ensures that the observations to controlling points are established before any outside influences have had an opportunity to degrade the setup. In making observations for an extended period of time at a particular instrument location, observe the control points from time to time. This ensures that any data observed between the control shots are good, or that a problem has developed and appropriate action can be taken to remedy the situation. As a minimum, require survey crews to observe both vertical and horizontal controls points at the beginning of each instrument setup and again before the instrument is picked up.

a. One of the major advantages of using a total station equipped with data collection is that errors previously attributed to blunders (i.e., transposition errors) can be eliminated. Even if the wrong reading is set on the horizontal circle in the field or the wrong elevation is used for the bench, the data itself may be precise. To make the data accurate, many software packages will allow the data to be rotated and/or adjusted as it is processed. The only way to assure that these corrections and/or observations have been accurately processed is to compare the data to control points. Without these observations in the magnetically recorded data, the orientation of that data will always be in question.

b. The use of a total station with a data collector can be looked upon as two separate and distinct operations. The checklists for setting up the total station and data collector are as follows:

(1) Total Station

(a) If EDM is modular, mount it on instrument.

(b) Connect data collector.

(c) Set up and level instrument.

(d) Turn on total station.

(e) Set atmospheric correction (ppm). This should be done in the morning and at noon.

(f) Set horizontal circle.

(g) Set coordinates.

(h) Observe backsight (check whether azimuth to backsight is 180 degrees from previous reading).

(i) Observe backsight benchmark (obtain difference in elevation). This may require factoring in the height of reflector above benchmark.

(j) Compute relative instrument height (benchmark elevation +/- difference in height). Note height of rod and note computations in field book.

(k) Input Z (elevation) value in instrument or data collector.

(l) Observe backsight benchmark (check elevation).

(m) Invert and repeat (check elevation).

(2) Data Collector

(a) Record date and job number.

(b) Record crew number and instrument serial number.

(c) Record field book number and page number.

(d) Record instrument location (coordinates).

(e) Record backsight azimuth.

(f) Record standard rod height.

(g) Record height of instrument.

Note: All the above information should also be recorded in the field book.

(h) Observe and record measurement to backsight benchmark.

(i) Enter alpha or numeric descriptor of above point into data collector.

(j) Observe and record measurement backsight benchmark or check benchmark (if setting benchmark, note in field book and repeat with instrument inverted).

(k) Enter alpha or numeric descriptor of above point into data collector.

(l) Observe and record measurement to backsight.

(m) Enter alpha or numeric descriptor of above point into data collector.

(n) Invert and repeat steps 12 and 13.

(o) Observe and record measurement to foresight.

(p) Enter alpha or numeric descriptor of above point into data collector.

(q) Invert and repeat steps 15 and 16.

(r) Observe and record measurement to side shot.

(s) Enter alpha or numeric descriptor of above point into data collector (repeat steps 18 and 19 as needed).

(t) When setup is complete, or at any appropriate time, repeat shots on vertical and horizontal control. Observe the displays and record in data collector.

4-27. Coding Field Data

Whether data are recorded by hand or electronically, one of the most time-consuming survey operations is the recording of a code or description to properly identify the point during processing. For example, in a topographic or planimetric survey, identification of points which locate the position of curbs, gutters, center lines, manholes, and other similar features are essential for their correct plotting and contour interpolation.

a. In spite of this slow coding process encountered when using today’s data collectors, the advantages heavily outweigh the disadvantages. These advantages include the collection of error-free numeric data from electronic total stations virtually at the instant they are available and the error-free transfer of these data to an office computer system without the need for manual entry.

b. Field coding allows the crew to perform the drafting and provide a more logical approach. Since the field crew can virtually produce the map from the field data this eliminates the need for many field book sketches. They can also eliminate office plotting, editing by connecting the dots, etc., to produce a final product. Total station users who gather data to be processed with other systems typically record descriptive information with each point measured, and gather 200 to 300 points per day with the total station. Users report 300 to 700 points per day if descriptive information is kept to the necessary minimum. The coding scheme is designed so the computer can interpret the recorded data without ambiguity to create a virtually finished product.

4-28. Field Computers

Many districts perform much of the survey reduction in the field. The greatest advantage of this procedure is uncovering a mistake which can easily be corrected if the crew and equipment are on the site. Laptop and notebook computers are popular field items. These computers are used to download GPS and total station data. Once the files are stored in the computer, data reduction can be done easily with programs stored in these machines.

a. Listed below are some software considerations to install on topographic field computers:

(1) Interface with field data collector.

(2) A system of predefined codes for most common objects and operations in a database.

(3) User-defined codes for site-specific requirements in a database.

(4) Survey adjustment programs such as:

- Compass rule adjustment

- Transit rule adjustment

- Crandell method
- Least squares
- Angle adjustment
- Distance adjustment

(5) Include a program which can assign an alpha-numeric descriptor field for each survey point.

(6) Include a full-screen editor to examine and edit ASCII data files.

(7) Have an interface program to convert files to common graphic interchange formats such as IGES or DXF.

(8) A program to connect features which were not recorded in order such as fence, curb and gutter, edge of pavement, waterline etc.

(9) Provide an operating system which will be compatible with post-processing machines with CADD programs such as Intergraph, Accugraph, AutoCAD.

(10) Custom programs which can use all the features available to the total station or the data collector.

(11) Select software which provides training if possible.

b. Requirements for field computer in data collection are:

(1) Portable.

(2) Ruggedized for field use.

(3) Processor: 80386 or 80486 must run MS-DOS 4.0 and greater.

(4) Memory: 640 kilobytes main memory (minimum), 40 megabyte internal hard disk (minimum, 80 megabyte recommended).

(5) Disk drives: 3½-inch floppy 720 k or 1.44 meg (1.44 preferred) external 5¼-inch floppy.

(6) Math coprocessor needed (must be compatible with main processor) (386).

(7) Serial port (RS 232), parallel port.

(8) Modem: 2400 baud autodial (Hayes compatible).

(9) VGA or Super VGA graphics.

(10) Portable ink-jet printer.

4-29. Modem for Data Transfer (Field to Office)

A modem is a device that modulates and demodulates binary data transmission over a telephone network. This device MODulates the carrier for transmission and DEModulates for reception; hence the term MODEM. The carrier is simply a tone with three characteristics, any one of which can be varied or modulated to impose a signal on the carrier. They include amplitude, frequency, and phase. The exact method of variation must be the same for two or more modems to be termed “compatible.”

4-30. Trigonometric Leveling and Vertical Traversing

Trigonometric leveling is the single most important new application brought into widespread use by the increasing acceptance of the total station. It is a fair statement that the error sources and types which affect trigonometric leveling measurements are among the least understood of the commonly done surveying procedures. See Table 4-2. A knowledge of the limitations of trigonometric leveling, together with means (instrumentation and procedures) to account for such limitations, is essential in using and supporting the use of modern surveying technology.

a. Total station trigonometric leveling can achieve accuracies similar to those reached using a spirit level. Third-Order accuracy should be easily obtainable. First-Order accuracy has been done, but the procedures are involved and not commonly followed.

b. Figures 4-8 and 4-9 depict the advantage of trigonometric leveling over that of spirit levels, especially high relief terrain.

Table 4-2
Elevation Errors Due to Errors in Zenith Angles. (Distances in Feet)

Sight Distance	Vertical Angle Uncertainty			
	1 Sec	5 Sec	10 Sec	60 Sec
100	0.0005	0.0024	0.005	0.03
200	0.0010	0.0048	0.010	0.06
400	0.0019	0.0097	0.019	0.12
500	0.0024	0.0121	0.024	0.15
1,000	0.0048	0.0242	0.049	0.29

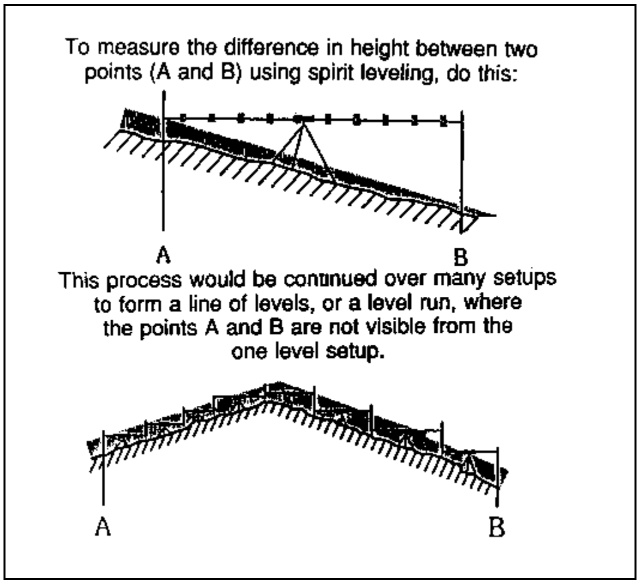

Figure 4-8. Spirit leveling

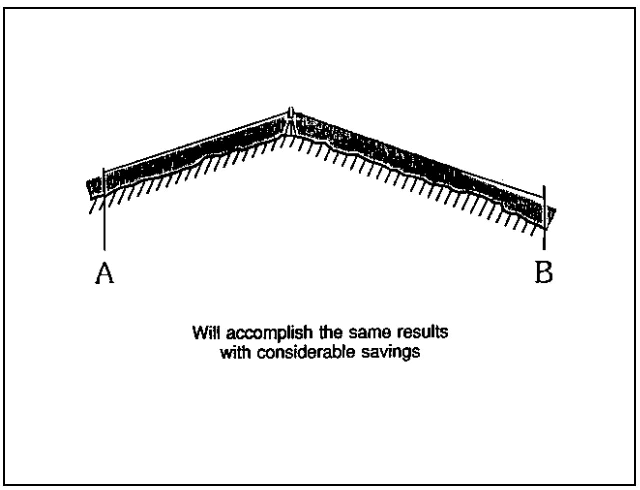

Figure 4-9. Trig leveling

4-31. Trigonometric Leveling Field Procedures

To obtain Third- or Second-Order vertical accuracies with a total station, the following field procedures should be rigorously followed:

- Careful setup and leveling
- Use Face I and Face II observations
- Reciprocal measurements
- Take multiple observations
- Protect instrument from sun and wind
- Use proper targetry based on Inst/EDM configuration
  - Tilting target if necessary
  - Good quality reflectors
  - Correct prism offsets
  - Unambiguous target
  - Maintain targetry in good adjustment
- Limited sight distances
  - 300 meters max
  - Reduce atmospheric-related error
  - Improves vertical angle accuracy
- Accurately measure temperature and pressure
  - At least twice a day
  - If long steep line measurements at both ends, use averages
- Watch for adverse refraction

4-32. Trigonometric Leveling Error Sources

The following error sources impact the accuracy of trigonometric leveling with electronic total stations:

- Instrument
  - Distances
  - Vertical angle accuracy
  - Vertical compensator important, dual axis compensation
  - No boost to vertical angle accuracy
- Nature
  - Curvature and refraction
  - Temperature/pressure correction
  - Wind, sun, and weather
- Operator
  - Accurate pointing
  - Lots of measurements
  - Reciprocal measurements
  - Measure HI and HT

Table 4-3 shows the precision resulting from horizontal distance and vertical angular measurements as needed to resolve differences in elevations from trigonometric observation.

Table 4-3
Combining Sources of Error (500-foot line)

Errors in Vertical Angle Measurement

Source	Type	Nominal Amount
Instrument accuracy	Random	+/- 3 sec
Collimation	Systematic	+/- 3 sec
Measure HI and HT	Random	+/-0.005 to 0.1 ft
C and R	Systematic	0.005 ft
Hand-held prism pole	Random	+/- 0.005 ft
30-mm prism offs error	Random	+/- 3 mm
Heatwaves	Random	+/- 0.01 ft
Unshaded instrument	Random	+/- 5 in. to 10 sec

Combined angular and linear error is between -0.024 and 0.048 foot. Vertical angle precision for the 500-foot line is therefore in the range 1:10,000 and 1:210,000.

Errors in Distance Measurement

Source	Type	Nominal Amount
Nominal accuracy	Random	+/-(5 mm + 5 ppm)
Temp estimation	Random	+/- 10 degrees F
Pressure estimation	Random	+/- 0.5 in Hg
Prism and instr cal.	Systematic	+/- 2 mm
Prism mispointing	Random	+/- 0.35 mm
Hand-held prism pole	Random	+/- 5 ft (fore/ft lean)

Combined error is between -0.0414 and +0.0546 foot. On this 500-ft line, this gives a range in precision of 1:9,000 to 1:12,000.