LS-002 Horizontal Control Survey Techniques

Instructor: J. Paul Guyer, P.E., R.A.
Course ID: LS-002
PDH Hours: 3 PDH
An Introduction to Horizontal Control Survey Techniques

J. Paul Guyer, P.E., R.A.
Editor

Paul Guyer is a registered civil engineer, mechanical engineer, fire protection engineer and architect with 35 years of experience designing buildings and related infrastructure. For an additional 9 years he was a principal staff advisor to the California Legislature on capital outlay and infrastructure issues. He is a graduate of Stanford University and has held numerous national, state and local offices with the American Society of Civil Engineers, Architectural Engineering Institute and National Society of Professional Engineers. He is a Fellow of ASCE, AEI and CABE (U.K.).
An Introduction to Horizontal Control Survey Techniques

J. Paul Guyer, P.E., R.A.
Editor

The Clubhouse Press
El Macero, California
CONTENTS

1. INTRODUCTION
2. TRADITIONAL HORIZONTAL CONTROL SURVEY TECHNIQUES
3. SECONDARY OR TEMPORARY HORIZONTAL CONTROL
4. BEARING AND AZIMUTH DETERMINATION
5. ELECTRONIC DISTANCE MEASUREMENT
6. COORDINATE COMPUTATIONS
7. TRAVERSE SURVEYS
8. TRAVERSE SURVEY GUIDELINES
9. TRAVERSE COMPUTATIONS AND ADJUSTMENTS
10. TRAVERSE ADJUSTMENT (COMPASS RULE)
11. TRIANGULATION AND TRILATERATION SURVEYS

(This publication is adapted from the Unified Facilities Criteria of the United States government which are in the public domain, have been authorized for unlimited distribution, and are not copyrighted.)
1. INTRODUCTION

1.1 PURPOSE AND SCOPE. Control surveys are performed to establish a monumented reference system for a facility mapping project. These fixed horizontal control points and vertical benchmarks are then used as starting points for supplemental topographic site plan mapping.

1.2 HORIZONTAL CONTROL SURVEY METHODS. Horizontal positions of permanent monuments around a facility or project site can be established by a number of survey techniques. These include traditional traverse, triangulation, or trilateration surveys from an established geodetic network on an installation or region (e.g., NSRS). Alternatively, GPS methods can be performed to extend control from an established network to the project site. Since most modern day survey crews or firms possess both GPS and total station equipment, there would be little justification for running lengthy (and costly) traverses or triangulation/trilateration networks to bring in control to a local site. Therefore, this discussion will focus on current practices for performing "traditional" horizontal control surveys--i.e. control being established using total station traverse methods. (This discussion does contain some background on older survey methods for use in basic surveying courses). Triangulation and trilateration methods will only be briefly addressed.

1.3 VERTICAL CONTROL SURVEY METHODS. As with horizontal control densification, a number of survey methods can be used to bring vertical control from an established datum into a project site. These include trigonometric leveling (e.g., a total station), differential (spirit) leveling, and differential GPS techniques. Since most facility mapping projects require fairly accurate elevations relative to a local network, traditional differential leveling is still the most effective and reliable method of transferring elevations. GPS elevation transfer methods are reliable over short distances; however, they are not as accurate as differential leveling methods.
2. TRADITIONAL HORIZONTAL CONTROL SURVEY TECHNIQUES

2.1 GENERAL OVERVIEW

2.1.1 PURPOSE. Horizontal control is established to serve as a basic framework for large mapping projects, to establish new horizontal control in a remote area, or to further densify existing horizontal control in an area.

2.1.2 INSTRUMENTS. Minimum instrument requirements for the establishment of primary control will typically include a repeating theodolite having an optical micrometer with a least-count resolution of six seconds (6") or better; a directional theodolite having an optical micrometer with a least count resolution of one arc-second; an EDM capable of a resolution of 1:10,000; or a total station having capabilities comparable to, or better than, any of the instruments just detailed. A calibrated 100-ft steel tape may also be used for measuring short distances.

2.1.3 MONUMENTATION. Primary project horizontal control points not permanently monumented in accordance with appropriate criteria and guidance should meet the following minimum standards:

2.1.3.1 MARKERS. Project horizontal control points should be marked with semi-permanent type markers (e.g., re-bar, railroad spikes, or large spikes). If concrete monuments are required, they will be set prior to horizontal survey work.

2.1.3.2 INSTALLATION. Horizontal control points should be placed either flush with the existing ground level or buried a minimum of one-tenth of a foot below the surface.

2.1.3.3 REFERENCE MARKS. Each primary control point should be referenced by a minimum of two points to aid in future recovery of that point. For this reference, well-defined natural or manmade objects may be used. The reference point(s) can be either set or existing and should be within 100 ft of the control point.
2.1.3.4 SKETCHES. A sketch should be placed in a standard field survey book or on a standard form, such as Figures 3-1 and 3-2. The sketch, at minimum, will show the relative location of each control point to the reference points and major physical features within 100 ft of the point.

2.1.4 REDUNDANCY. A minimum of two repeated angle measurements (i.e. positions or sets) should be made for establishing project control points. With EDM distance measurements, a minimum of two readings should be taken at each setup and recorded in a standard field book (or data collector). The leveled height of the instrument and the height of the reflector should be measured carefully to within 0.01 ft and recorded. Each measured slope distance (taped or EDM) should be reduced to a horizontal distance using either reciprocal vertical angle observations or the known elevation of each point obtained from differential leveling. Duplicate distances should be observed over each line by remeasuring backsight lines at each traverse point set up. Depending on the accuracy requirements, additional sets of angle measurements or EDM distances may be specified.
The station is located on Stocho Barracks, Illsheim, Germany.

To reach the station front gate of Stocho Barracks (Grid 0082), go straight for 0.1 mile to four-way intersection. Turn right (west) and proceed 0.8 mile to the gate of the access road and a guard shack. Follow the access road around the perimeter of the airfield for 0.9 mile to the station site.

The station is a Type 70 monument protruding 20 cm above the ground and is located atop a berm.

The station is located 75.1 m at an azimuth of 160° from Building 6680, 82.3 m from the hot fuel point and 57 m from the fuel point sign.

Horizontal position was established by third-order class I traverse.

Elevation was established by third-order leveling procedures.

---

**Figure 3-1.**

Form -- Description or Recovery of Horizontal Control Station
<table>
<thead>
<tr>
<th>COUNTRY</th>
<th>TYPE OF MARK</th>
<th>STATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>USA</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LOCALITY</th>
<th>STAMPING ON MARK</th>
<th>AGENCY (CAST IN MARKS)</th>
<th>ELEVATION (FT)</th>
<th>LATITUDE</th>
<th>LONGITUDE</th>
<th>DATUM</th>
<th>DATUM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Corps of Engineers</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(NORTING)</th>
<th>(EASTING)</th>
<th>GRID AND ZONE</th>
<th>ESTABLISHED BY</th>
<th>(NORTING)</th>
<th>(EASTING)</th>
<th>GRID AND ZONE</th>
<th>DATE</th>
<th>ORDER</th>
</tr>
</thead>
<tbody>
<tr>
<td>FT</td>
<td>FT</td>
<td></td>
<td></td>
<td>M</td>
<td>M</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

To obtain grid azimuth, add * ' ' ' ' ' ' ' ' ' ' to the geodetic azimuth.

To obtain grid azimuth (add/sub) * ' ' ' ' ' ' ' ' ' ' to the geodetic azimuth.

<table>
<thead>
<tr>
<th>OBJECT</th>
<th>AZIMUTH OR DIRECTION (GEODETIC) (GRID) (MAGNETIC)</th>
<th>BACK AZIMUTH</th>
<th>GEOD DISTANCE (METERS) (FEET)</th>
<th>GRID DISTANCE (METERS) (FEET)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3-2
Blank Form 1959 -- Description or Recovery of Horizontal Control Station
2.1.5 **REPEATING THEODOLITE.** If a repeating theodolite (e.g., a Wild T1) is used for the horizontal angles, the instrument will be pointed at the backsight station with the telescope in a direct reading position, and the horizontal vernier set to zero degrees. All angles should then be turned to the right, and the first angle recorded in a field book. The angle should be repeated a minimum of four times (i.e. two sets) by alternating the telescope and pointing in the direct and inverted positions. The last angle will also be recorded in the field book. If the first angle deviates more than five seconds (5") from the result of the last angle divided by four, the process should be repeated until the deviation is less than or equal to five seconds. Multiples of 360 degrees may need to be added to the last angle before averaging. The horizon should be closed by repeating this process for all of the sights to be observed from that location. The foresight for the last observation should be the same as the backsight for the first observation. If the sum of all the angles turned at any station deviates more than ten seconds (10") from 360 degrees, the angles should be turned again until the summation is within this tolerance.

2.1.6 **DIRECTIONAL THEODOLITE.** If a directional theodolite (e.g., Wild T2 or Wild T3) is used for the horizontal angles, the instrument should be pointed at the backsight station with the telescope in a direct reading position and the horizontal scales set to within ten seconds (10") of zero degrees. The scales should be brought into coincidence and the angle read and recorded in the field book. The angles (directions) should then be turned to each foresight in a clockwise direction, and the angles read and recorded in a field book. This process will continue in a clockwise direction and should include all sights to be observed from that station. The telescope should then be inverted and the process repeated in reverse order, except the scales are not to be reset, but will be read where it was originally set. The angles between stations may then be computed by differencing the direct and reverse readings. This process of observing a "set" should be repeated two or more times, depending on the survey specification. It is difficult to set the angle values precisely on the plates of an optical theodolite. Angles are determined by reading the initial and the final directions, and then determining the angular difference between the two directions. Optical theodolites are generally very precise--a Wild T2 optical theodolite reads directly to 1". If several sets are required for precision.
purposes, distribute the initial settings around the plate circle to minimize the effect of circle-gradation distortions.

2.1.7 HORIZONTAL DISTANCES. To reduce EDM slope distances to horizontal, a vertical angle observation must be obtained from each end of each line being measured. The vertical angles should be read in both the direct and inverted scope positions and adjusted. If the elevations for the point on each end of the line being measured are obtained by differential leveling, then this vertical angle requirement is not necessary.

2.1.8 TARGETS. All targets established for backsights and foresights should be fixed and centered directly over the measured point. Target sights may be a reflector or other type of target set in a tribrach, a line rod plumbed over the point in a tripod, or guyed/fixed in place from at least three positions. Artificial sights (e.g., a tree on the hill behind the point) or hand held sights (e.g., line rod or plumb bob string) should not be used to set primary control targets.

2.1.9 CALIBRATION. All theodolites, total stations, EDM, and prisms used for horizontal control work should be serviced regularly and checked frequently. Tapes and EDMs must be periodically calibrated over lines of known length, such as NGS calibration baselines. Instrument calibrations should be done at least annually. Theodolite instruments should be adjusted for collimation error at least once a year and whenever the difference between direct and reverse reading of any theodolite deviates more than thirty seconds from 180 degrees. Readjustment of the cross hairs and the level (plate) bubble should be done whenever misadjustments affect the instrument reading by more than the least count of the reading scales of the theodolite. Forced centering type tribrachs should be periodically (monthly) checked to ensure the optical plumb line is correct. Circular or "bulls eye" bubbles on tribrachs, total stations, rods, etc. should be periodically checked and adjusted. Tribrach or total station optical plummets (visual or laser) must be periodically checked.
2.1.10 HORIZONTAL DIRECTION RECORDING. Procedures for recording horizontal directions are the same for all orders of accuracy. Record horizontal directions in a bound field survey book (see Figure 3-3 below), or any equivalent electronic recording form. Each time a point is occupied, the following information should be recorded--either on the Title Page or entry page, as appropriate:

Title Page:
• Instrument make, model, and serial number.
• Instrument operator’s name.
• Recorder’s name.
• Weather description.
  o Temperature.
  o General atmospheric condition.
  o Wind.

Entry Page:
• Designation of the occupied station.
  o Full station name.
  o Year established.
  o Name of the agency on the disk.

The field book or recording form should include the above information for each station observed. If an instrument, signal, or target is set eccentric to a station (not plumbed directly over the station mark), that item should be sketched on the recording form. The sketch should include the distance and the directions that the eccentric item is from the station. When intersection stations are observed, the exact part of the point observed must be recorded and shown on the sketch.

2.1.11 HORIZONTAL ABSTRACTS OF DIRECTIONS. An abstract of horizontal directions should be compiled for every station at which horizontal directions have been observed. An appropriate form or equivalent field book abstracts should be completed before leaving the point. If a horizon closure is specified, the corrected station angle and
the corrected explement angle should be recorded in the field book before leaving the point. If a form is used, readings will be entered opposite the proper circle position, as indicated in the field notes. The degrees and minutes for each direction are entered one time at the top of each column, and the seconds are entered for each circle position.

<table>
<thead>
<tr>
<th>Object</th>
<th>D/R</th>
<th>Deg</th>
<th>Min</th>
<th>Sec</th>
<th>Mean</th>
<th>Diff°</th>
<th>Date/Notes/Packer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jerry</td>
<td>D</td>
<td>00</td>
<td>00</td>
<td>15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1980</td>
<td>R</td>
<td>180</td>
<td>00</td>
<td>10</td>
<td>12.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Center</td>
<td>D</td>
<td>95</td>
<td>48</td>
<td>39</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>R</td>
<td>275</td>
<td>48</td>
<td>33</td>
<td>36.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pat, 82</td>
<td>D</td>
<td>196</td>
<td>22</td>
<td>06</td>
<td></td>
<td></td>
<td>23.5 95° 48'</td>
</tr>
<tr>
<td></td>
<td>R</td>
<td>16</td>
<td>22</td>
<td>02</td>
<td>04.0</td>
<td></td>
<td>51.5 196° 21'</td>
</tr>
<tr>
<td>9 Jerry</td>
<td>R</td>
<td>270</td>
<td>02</td>
<td>38</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>90</td>
<td>02</td>
<td>43</td>
<td>40.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Center</td>
<td>R</td>
<td>15</td>
<td>50</td>
<td>56</td>
<td></td>
<td></td>
<td>17.0 95° 48'</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>185</td>
<td>50</td>
<td>59</td>
<td>57.5</td>
<td></td>
<td>53.5 196° 21'</td>
</tr>
<tr>
<td>Pat, 82</td>
<td>R</td>
<td>106</td>
<td>24</td>
<td>32</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>286</td>
<td>24</td>
<td>36</td>
<td>34.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 3-3**
Sample horizontal field book recording--Directional Theodolite
3. SECONDARY OR TEMPORARY HORIZONTAL CONTROL

3.1 GENERAL. Secondary horizontal control is established to determine the location of structure sections, cross sections, or topographic features, for construction control, or to pre-mark requirements for small to medium scale photogrammetric mapping. These points are often temporary in nature and can easily be reset from the permanent primary control points.

3.2 REQUIREMENTS. Secondary horizontal control requirements are identical to that described for primary horizontal control with the following exceptions.

3.2.1 MONUMENTATION. It is not required for secondary horizontal control points to have two reference points. Wooden hubs, PK nails, or other similar markings are adequate. Descriptions or sketches are usually not required.

3.2.2 WHEN A TOTAL STATION OR EDM IS USED, a minimum of two readings should be taken at each setup and recorded in a standard field book or electronic data collector.

3.2.3 IF A REPEATING THEODOLITE is used for the horizontal angles, the angle measurement should be repeated a minimum of two times by alternating the telescope and pointing in the direct and inverted positions.

3.2.4 IF A DIRECTIONAL THEODOLITE is used for the horizontal angles, the process (described for primary control) should be repeated two times--for a total of two data set collections.
4. BEARING AND AZIMUTH DETERMINATION. Horizontal angles are usually turned (or deflected) to the right or left. The three types of angle measurements are as follows:

- Interior angles. If angles in a closed figure are to be measured, the interior angles are normally read. When all interior angles have been recorded, the accuracy of the work can be determined by comparing the sum of the abstracted angles with the computed value for the closed loop (Figure 3-4 below).

- Deflection angles. In an open traverse (Figure 3-4), the deflection angles are measured from the prolongation of the backsight line to the foresight line. The angles are measured either to the left or to the right. The direction must be shown along with the numerical value.

- Vertical angles. Vertical angles can be referenced to a horizontal or vertical line (Figure 3-5).

Optical-micrometer theodolites measure vertical angles from the zenith (90° or 270° indicate a horizontal line). Zenith and nadir are terms describing points on a sphere. The zenith point is directly above the observer, and the nadir point is directly below the observer. The observer, the zenith, and the nadir are on the same vertical line.
Figure 3-4

Interior angles on a closed traverse (top) and deflection angles on an open traverse (bottom)
4.1 BEARING TYPES. The bearing of a line is the direction of the line with respect to a given meridian. A bearing is indicated by the quadrant in which the line falls and the acute angle that the line makes with the meridian in that quadrant. Observed bearings are those for which the actual bearing angles are measured, while calculated bearings are those for which the bearing angles are indirectly obtained by calculations. A true bearing is made with respect to the astronomic north reference meridian. A magnetic bearing is one whose reference meridian is the direction to the magnetic poles. The location of the magnetic poles is constantly changing; therefore the magnetic bearing between two points is not constant over time. The angle between a true meridian and a magnetic meridian at the same point is called its magnetic declination. An assumed bearing is a bearing whose prime meridian is assumed. The relationship between an assumed bearing and the true meridian should be defined, as is the case with most SPCS grids.
4.2 BEARING DETERMINATION GUIDELINES. All bearings used for engineering applications should be described by degrees, minutes, and seconds in the direction in which the line is progressing. Bearings are recorded with respect to its primary direction, north or south, and next the angle east or west. For example, a line can be described as heading north and deflected so many degrees east or west. Alternatively, a line also can be described as heading south and deflected so many degrees east or west. A bearing will never be listed with a value over 90 degrees (i.e. the bearing value always will be between over 0 degrees and 90 degrees. Bearing angles are computed from a given azimuth depending on the quadrant in which the azimuth lies. When the azimuth is in the first quadrant (0° to 90°), the bearing is equal to the azimuth. When the azimuth is in the second quadrant (90° to 180°), the bearing is equal to 180° minus the azimuth. When the azimuth is in the third quadrant (180° to 270°), the bearing is equal to the azimuth minus 180°. When the azimuth is in the fourth quadrant (270° to 360°), the bearing is equal to 360° minus the azimuth. Since the numerical values of the bearings repeat in each quadrant, the bearings must be labeled to indicate which quadrant they are in. The label must indicate whether the bearing angle is measured from the north or south line and whether it is east or west of that line. For example, a line with an azimuth of 341° 12' 30" falls in the fourth or northwest (NW) quadrant and its bearing is N 18° 47' 30" W.

4.3 AZIMUTH TYPES. The azimuth of a line is its direction as given by the angle between the meridian and the line, measured in a clockwise direction. Azimuths can be referenced from either the south point or the north point of a meridian. (Geodetic azimuths traditionally have been referenced to the south meridian whereas grid azimuths are referenced to the north meridian). Assumed azimuths are often used for making maps and performing traverses, and are determined in a clockwise direction from an assumed meridian. Assumed azimuths are sometimes referred to as "localized grid azimuths." Azimuths can be either observed or calculated. Calculated azimuths consist of adding to or subtracting field observed angles from a known bearing or azimuth to determine a new bearing or azimuth.
4.4 AZIMUTH DETERMINATION GUIDELINES. Azimuths will be determined as a line with a clockwise angle from the north or south end of a true or assumed meridian. For traverse work using angle points, the traverse closure requirements outlined in Chapter 4 will be followed.

4.5 ASTRONOMIC AZIMUTH. In order to control the direction of a traverse, an astronomic azimuth must be observed at specified intervals and abrupt changes of direction of the traverse. Astronomic azimuth observations can be made by the well-known hour angle or altitude methods. Azimuth observations should be divided evenly between the backsight and foresight stations as reference objects. Using the rear station, turn clockwise to forward station then to star, reverse telescope on star, then forward station and back to rear station. Then using forward station, turn clockwise to rear station then to star, reverse telescope on star, then rear station and back to forward station. The number of position repetitions will depend upon the order of accuracy required. Since GPS has effectively eliminated the need for lengthy traverse networks, astronomic azimuth observations are rarely ever required. Exceptions may involve boundary surveys originally referenced from solar azimuth observations. Procedures for observing astronomic azimuths can be found in the references listed at Appendix A-2. (Note that GPS azimuths determined relative to WGS 84 must be corrected to the reference orientation of the local datum).
5. ELECTRONIC DISTANCE MEASUREMENT. The distance between two points can be horizontal, slope, or vertical. A tape measure or an EDM device (such as a total station) can measure horizontal and slope distances. A distance measured on a slope can be trigonometrically converted to its horizontal equivalent by using the slope angle or vertical difference of elevation (DE). Figure 3-6 below illustrates a basic example of the geometry used to determine the horizontal distance of a measurement over uneven ground.

![Geometry of an EDM measurement](image)

**Figure 3-6**
Geometry of an EDM measurement

Alternatively, the elevations of the occupied hubs (Stations A and B in Figure 3-6 above) may have been determined by differential levels. Applying the measured HI and HT yields the absolute elevation of the instrument and target. The measured slope distance "S" can then be reduced to a horizontal distance "H" given the delta elevation between the instrument and target. A meteorological correction is applied to the observed slope distance before reducing it to horizontal. Subsequently, the horizontal distance is corrected for grid scale and sea level. A traditional field book example of a horizontal slope distance observation is shown in Figure 3-7 below. In this example, slope distances are manually recorded along with meteorological data. A series of 10 slope
distances were observed and averaged. A meteorological correction is applied along with a constant instrument/system constant. The resultant slope distance "T" (76.106 m) is reduced to horizontal, then to a grid distance (Hg in Figure 3-7). No sea level correction was applied since this project was set on an arbitrary datum (PICES). Note that Figure 3-7 illustrates the internal computations now automatically performed in a total station/data collector system.

Figure 3-7
Horizontal distance observations and reductions--manual computations in field book

5.1 ERRORS. Distances measured using an EDM are subject to the same errors as direction measuring equipment. The errors also include instrumental component errors. Instrumental errors are usually described as a number of millimeters plus a number of ppm. The accuracy of the infrared EDM is typically ± (5 millimeters + 5 ppm). The ppm
accuracy factor can be thought of in terms of millimeters per kilometer, as there are 1 million millimeters in 1 kilometer. This means that 5 ppm equals 5 millimeters per kilometer. Errors introduced by meteorological factors must be accounted for when measuring distances of 500 meters or more. Accurate ambient temperature and barometric pressure must be measured. An error of 1 degree Celsius (C) causes an error of 0.8 ppm for infrared distances. An error of 3 millimeters of mercury causes an error of 0.9 ppm in distance.

5.2 INSTRUMENT CONSTANTS. Although manufacturers provide instrument and prism constants, it is essential that instrument constants be verified under actual operating conditions, especially for precise surveys. The following factors must be considered:

- The use of a prism typically provides an indicated distance longer than the true value. Applying a negative correction will compensate for this effect. Each prism should have its own constant or correction determined individually, and a master file should be maintained.
- An instrument constant can be either positive or negative and may change due to the phase shifts in the circuitry. Therefore, a positive or a negative correction may be required.
- The algebraic sum of the instrument and the prism constants are referred to as the total constant. The correction for the total constant (equal in magnitude but opposite in sign) is referred to as the total constants correction, from which the instrument or prism constant can be computed if one or the other is known.
6. COORDINATE COMPUTATIONS. If the coordinate of a point and the azimuth (or bearing) and distance from that point to a second point are known, the coordinate of the second point can be computed. In Figure 3-8 below, the azimuth and distance from Station A to Station B are determined by measuring the horizontal angle (β) from the azimuth mark to Station B and the distance from Station A to Station B.

![Forward Position Computation Diagram]

Figure 3-8
Forward Position Computation

The azimuth (or bearing) from A to B (α) is determined by reducing the observed azimuth to the relative quadrant. For example, in Figure 3-8, if the azimuth from Point A to the Azimuth Mark is 320°, and observed angle "β" from Station A between the reference azimuth point and Point B is 105°, then the azimuth of the line from Point A to Point B "α" is computed from:

\[
\text{Azimuth (α) from } A \rightarrow B = 105° - (360° - 320°) = 65° \text{ [or bearing N 65° E]}
\]
The computation of the difference in northing (dN) and the difference in easting (dE) requires the computation of a right triangle. The distance from Station A to Station B ("s" in Figure 3-8--reduced to horizontal, sea level, corrected for grid scale, etc.) is the hypotenuse of the triangle, and the bearing angle (azimuth) is the known angle. The following formulas are used to compute dN and dE:

\[
\begin{align*}
  dN &= s \cdot \cos(\alpha) \\
  dE &= s \cdot \sin(\alpha)
\end{align*}
\]

(Eq 3-1)

If the traverse leg falls in the first (northeast [NE]) quadrant, the value of the easting increases as the line goes east and the value of the northing increases as it goes north. The product of the dE and the dN are positive and are added to the easting and northing of Station A to obtain the coordinate of Station B, as shown in Figure 3-8. When using trigonometric calculators to compute a traverse, enter the azimuth angle, and the calculator will provide the correct sign of the function and the dN and the dE. If the functions are taken from tables, the computer provides the sign of the function based on the quadrant. Lines going north have positive dNs; lines going south have negative dNs. Lines going east have positive dEs; lines going west have negative dEs. The following are examples of how to compute the dN and the dE for different quadrants:
• Given an azimuth from Station A to Station B of 70° 15' 15" and a distance of 568.78 meters (this falls in the first [NE] quadrant), compute the dN and the dE.

\[ \text{dN} = \cos 70° 15' 15" \cdot 568.78 = +0.337848 \cdot 568.78 = +192.16 \text{ m} \]

\[ \text{dE} = \sin 70° 15' 15" \cdot 568.78 = +0.941200 \cdot 568.78 = +535.34 \text{ m} \]

• Given an azimuth from Station B to Station C of 161° 12' 30" and a distance of 548.74 meters (this falls in the second [southeast] [SE] quadrant), compute the dN and the dE.

\[ \text{dN} = \cos 161° 12' 30" \cdot 548.74 = -0.946696 \cdot 548.74 = -519.49 \text{ m} \]

\[ \text{dE} = \sin 161° 12' 30" \cdot 548.74 = +0.322128 \cdot 548.74 = +176.76 \text{ m} \]

• Given an azimuth from Station C to Station A of 294° 40' 45" and a distance of 783.74 meters (this falls in the fourth [NW] quadrant), compute the dN and the dE.

\[ \text{dN} = \cos 294° 40' 45" \cdot 783.74 = +0.417537 \cdot 783.74 = +327.24 \text{ m} \]

\[ \text{dE} = \sin 294° 40' 45" \cdot 783.74 = -0.908660 \cdot 783.74 = -712.15 \text{ m} \]
7. TRAVERSE SURVEYS. A traverse survey is defined as the measurement of the lengths and directions of a series of straight lines connecting a series of points on the earth. Points connected by the lines of a traverse are known as traverse stations. The measurements of the lengths and directions are used to compute the relative horizontal positions of these stations. Traversing is used for establishing basic area control where horizontal positions of the traverse stations, and elevations of the stations, must be determined. If reference azimuth marks or features are not available, astronomic observations and/or GPS-derived azimuths are made along a traverse at prescribed intervals to control the azimuth alignment of the traverse. The interval and type of controlling azimuth observation will depend upon the order of accuracy required and the traverse methods used; and the availability of existing control.

7.1 TRAVERSE TYPES. There are two basic types of traverses, namely, closed traverses and open traverses.

7.1.1 CLOSED TRAVERSE. A traverse that starts and terminates at a station of known position is called a closed traverse. The order of accuracy of a closed traverse depends upon the accuracy of the starting and ending known positions and the survey methods used for the field measurements. There are two types of closed traverses.
7.1.1.1 LOOP TRAVERSE. A loop traverse starts on a station of known position and terminates on the same station—e.g., Station A in Figure 3-9 above. An examination of the position misclosure in a loop traverse will reveal measurement blunders and internal loop errors, but will not disclose systematic errors or external inaccuracies in the control point coordinates. In a loop traverse, the measured angular closure is the summation of the interior or exterior horizontal angles in the traverse. If there are "n" sides in a loop traverse, and interior angles were measured, the true angular closure should equal (n-2) \( \cdot 180^\circ \). If exterior angles were measured when performing a loop traverse, the true angular closure should equal (n+2) \( \cdot 180^\circ \). In Figure 3-9 above, the starting azimuth from Station "A" is not shown. This initial azimuth might have been taken from a GPS,
magnetic, or astronomic observation--or even an arbitrary (assumed) value might have been used.

7.1.1.2 CONNECTING TRAVERSE. A connecting traverse starts on a station of known position and terminates on a different station of known position. An example would be Stations "A" and "B" in Figure 3-9 above--if these two points have fixed coordinates (and azimuth A-B between them). When using this type of traverse, the systematic errors and position inaccuracies can be detected and eliminated along with blunders and accidental errors. The ability to correct measurement error depends on the known accuracy of the control point coordinates, and related azimuth references used at each end of the traverse.

7.1.2 OPEN TRAVERSE. (Figure 3-10 below). An open traverse starts on a station of known position and terminates on a station of unknown position. With an open traverse, there are no checks to determine blunders, accidental errors, or systematic errors that may occur in the measurements. The open traverse is very seldom used in topographic surveying because a loop traverse can usually be accomplished with little added expense or effort.
7.2 RIGHT-OF-WAY TRAVERSE. A right-of-way traverse normally starts and ends on known points. This type of traverse can be run with a transit and steel tape, EDM, or total station. The style of notes is similar to most traverses with the only difference being the type of detail shown. Fences can be of particular importance in determining right-of-way limits, especially when working in an area not monumented. Notes for right-of-way traverses should be especially clear and complete for many times this type of traverse is the basis for legal or court hearings regarding true property corners. If a search for a corner is made and nothing is found, a statement should be written in the field book to this effect. Property title searches and deed research will generally be required to obtain appropriate existing descriptions, plans, and other documents, which are generally available in the public record.

7.3 STADIA TRAVERSE. Uses of stadia traverses include rough or reconnaissance type surveys, checking on another traverse for errors, and control for a map being made by stadia methods on a very large scale. Stadia traverses are rarely performed given the availability of total stations today.
7.4 COMPASS TRAVERSE. A compass traverse is made to establish the direction of a line by magnetic compass measurements (i.e. no angles are turned). Distances are usually measured by stadia or paced. These types of surveys are rarely performed.
8. TRAVERSE SURVEY GUIDELINES. Several basic steps are required to plan and execute a traverse survey:

- research existing control in the project area
- design survey to meet specifications
- determine types of measurements
- determine types of instruments
- determine field procedures
- site reconnaissance and approximate surveys
- install monuments and traverse stations
- data collection
- data reduction
- data adjustment
- prepare survey report

8.1 THE FOLLOWING GENERAL GUIDELINES are recommended in performing traverse surveys:

8.1.1 PREPARATION. For most applications, it is recommended that permanent points be established at intervals of one mile or less, starting at a known point--preferably a NGS published control point on the NSRS. Plan the traverse to follow a route that will be centered as much in the project area as possible, and avoiding areas that will be affected by construction, traffic, or other forms of congestion. The route should provide a check into other known points as often as practicable. After determining the route, it is best to set temporary or permanent monuments (e.g., wooden hubs, PK nails, iron rods, brass caps in concrete, or some other suitable monument) at each angle point on the traverse. Ensure there is a clear line of sight from angle point to angle point and determine an organized numbering or naming system to mark all points when set.

8.1.2 ACCURACY REQUIREMENTS. Control traverses are run for use in connection with all future surveys to be made in the area of consideration. They may be of Second,
Third, or Fourth-Order accuracy, depending on project requirements. Most project requirements will be satisfied with Second- or Third-Order accuracies. The order of accuracy for traversing may also be determined by the equipment and methods used to collect the traverse measurements, by the final accuracy attained, and by the coordinate accuracy of the starting and terminating stations of the traverse. The point closure standards indicated in Chapter 4 must be met for the appropriate accuracy classification to be achieved.

8.1.3 POSITION AND AZIMUTH ORIENTATION. If it is impossible to start or terminate on stations of known position and/or azimuth, then a GPS or astronomic observation for position and/or azimuth must be conducted. Astronomic position or azimuth observations are no longer practical given the ease of GPS for these requirements. Two GPS static points can be established at the ends of a traverse, from which a starting position and azimuth is available. The GPS azimuth point should be 500 to 1,000 ft distant from the initial point. Extreme care should be taken not to mix up astronomic, geodetic, GPS, magnetic, and grid azimuths--they are all different.

8.1.4 TRAVERSE ROUTE. The specific route of a new traverse should be selected with care, keeping in mind its primary purpose and the flexibility of its future use. Angle points should be set in protected locations if possible. Examples of protected locations include fence lines, under communication or power lines, near poles, or near any permanent concrete structure. It may be necessary to set critical points below the ground surface. If this is the case, reference the traverse point relative to permanent features by a sketch, as buried points are often difficult to recover at future dates. Select sites for traverse stations as the traverse progresses. Locate the stations in such a way that, at any one station, both the rear and forward stations are visible. The number of stations in a traverse should be kept to a minimum to reduce the accumulation of instrument errors and the amount of computing required. Short traverse legs (courses or sections) require the establishment and use of a greater number of stations and may cause excessive errors in the azimuth. Small errors in centering the instrument, in
station-marking equipment, and in instrument pointings, can be magnified over short courses and can result in abnormally high azimuth closures.

8.1.5 TEMPORARY HUBS. Temporary station markers are usually 2x2-inch wooden hubs, 6 inches or more in length. These hubs should be driven flush with the ground, especially in maintained areas or where the hubs could present a hazard. The center of the top of the hub is marked with a surveyor’s tack or an "X" to designate the exact point of reference for angular and linear measurements. To assist in recovering a station, a reference stake (e.g., a flagged 1 x 2 inch wood stake) may be set near the hub. The reference stake should be marked with the traverse station designation, stationing, offset, etc.—as applicable.

8.1.6 MEASUREMENTS. Follow manufacturer instructions for operation of theodolites, EDM, or total stations. When using an EDM or total station, a minimum of two redundant readings should be made before moving to the next occupation point. Special care should be taken with the type of sights used for angle measurement—fixed rigid sights should be used, not hand held targets on poles. For directional theodolite or total station angle measurements, at least two sets (positions) of angles should be made. Always measure horizontal angles at the occupied station by sighting the instrument at the rear station and measuring the clockwise angles to the forward station. A horizon closure may be performed as a check.

8.1.7 FIELD DATA REDUCTIONS. All survey field notes should be carefully and completely reduced; with the mean angle calculated in the field and recorded along with the sketch. All traverse adjustments should be made in the office unless this capability is available on the data collector in the field. A sketch of the permanent monument locations should be made in the field and a detailed description on how to recover them should be recorded in writing. This information can be used for making subsequent record of the survey monument and survey report. Temporary monuments need only be briefly described in the field notes.
9. TRAVERSE COMPUTATIONS AND ADJUSTMENTS. There are a number of methods available for adjusting traverses. The most common are listed below.

9.1 CRANDALL RULE. The Crandall rule is used when the angular measurements (directions) are believed to have greater precision than the linear measurements (distances). This method allows for the weighting of measurements and has properties similar to the method of least squares adjustment. Although the technique provides adequate results, it is seldom utilized because of its complexity. In addition, modern distance measuring equipment and electronic total stations provide distance and angular measurements with roughly equal precision. Also, a standard Least Squares adjustment can be performed with the same amount of effort.

9.2 COMPASS RULE. The Compass Rule adjustment (also called the Bowditch Method) is used when the angular and linear measurements are of equal precision. This is the most widely used traverse adjustment method. Since the angular and linear precision are considered equivalent, the angular error is distributed equally throughout the traverse. For example, the sum of the interior angles of a five-sided traverse should equal 540° 00’ 00".0, but if the sum of the measured angles equals 540° 01’ 00".0, a value of 12".0 must be subtracted from each observed angle to balance the angles within traverse. After balancing the angular error, the linear error is computed by determining the sums of the north-south latitudes and east-west departures. The misclosure in latitude and departure is applied proportional to the distance of each line in the traverse.

9.3 LEAST SQUARES. The method of least squares is the procedure of adjusting a set of observations that constitute an over-determined model (redundancy > 0). A least squares adjustment relates the mathematical (functional model) and stochastic (stochastic model) processes that influence or affect the observations. Stochastic refers to the statistical nature of observations or measurements. The least squares principle relies on the condition that the sum of the squares of the residuals approaches a minimum.
9.3.1 FUNCTIONAL MODEL. The functional model relates physical or geometrical conditions to a set of observations. For example, if a surveyor measures the interior angles of a five-sided figure, the sum of these angles should add up to 540°. If the correct model is not determined, the adjusted observations will be in error.

9.3.2 STOCHASTIC MODEL. The stochastic model is the greatest advantage of the least squares procedure. In least squares adjustment, the surveyor can assign weights, variances, and covariance information to individual observations. The traditional traverse balancing techniques do not allow for this variability. Since observations are affected by various errors, it is essential that the proper statistical estimates be applied.

9.3.3 OBSERVATIONS. Observations in least squares are the measurements that are to be adjusted. An adjustment is not warranted if the model is not over-determined (redundancy = 0). Observations vary due to blunders and random and systematic errors. When all blunders and systematic errors are removed from the observations, the adjustment provides the user an estimate of the “true” observation.

9.3.4 BLUNDERS. Blunders are the result of mistakes by the user or inadvertent equipment failure. For example, an observer may misread a level rod by a tenth of a foot or a malfunctioning data recorder may cause erroneous data storage. All blunders

\[ v'wv \rightarrow \text{minimum} \]  

(Eq 3-2)

where
- \( v \) = observation residual
- \( w \) = weight of observation

The residuals (\( v \)) are the corrections to the observations. The final adjusted observations equal the observation plus the post-adjustment residual.

\[ \hat{l} = l + v \]  

(Eq 3-3)

where
- \( \hat{l} \) = adjusted observation
- \( l \) = observation
- \( v \) = observation residual
must be removed before the least squares adjustment procedure. Blunders can be identified by scrutinizing the data before they are input in the adjustment software. Preliminary procedures like loop closures, traverse balancing, and weighted means are techniques that can identify blunders before adjustment.

9.3.5 SYSTEMATIC ERRORS. Systematic errors are the result of physical or mathematical principles. These errors must be removed before the adjustment procedure. Systematic errors are reduced or eliminated through careful measurement procedures. For example, when using a total station EDM, the user should correct the distance for meteorological effects (temperature, pressure, relative humidity).

9.3.6 RANDOM ERRORS. Random errors are an unavoidable characteristic of the measurement process. The theories of probability are used to quantify random errors. The theory of least squares is developed under the assumption that only random errors exist within the data. If all systematic errors and blunders have been removed, the observations will differ only as the result of the random errors.

9.3.7 REFERENCES. Many field data collectors are capable of performing Least Squares traverse adjustments; thus, simple traverses are more frequently being adjusted by this method.
10. TRAVERSE ADJUSTMENT (COMPASS RULE). The Compass Rule is a simple method and is most commonly employed for engineering, construction, and boundary surveys. It is also recognized as the accepted adjustment method in some state minimum technical standards. The following sections only briefly describe traverse adjustment techniques—detailed procedures and examples of traverse adjustments can be found in many of the texts.

10.1 GENERAL. Traverse computations and adjustments require the following steps:

- Adjust angles and directions to fixed geometric conditions based on angular misclosure
- Calculate latitudes (dY or dN) and departures (dX or dE) of the traverse misclosure
- Distribute the misclosure latitudes and departures over the traverse
- Compute adjusted coordinates of the traverse stations
- Calculate final adjusted lengths and azimuths between traverse points

10.2 ANGLE COMPUTATIONS AND ADJUSTMENTS. The azimuth of a line is the horizontal angle (measured clockwise) from a base direction to the line in question. To compute a traverse, surveyors determine the azimuth for each traverse leg, starting with the fixed azimuth at the known starting point. This fixed azimuth is typically that computed between the fixed starting station and some azimuth reference point (another monument, a known object, or astronomical), as was shown back on Figure 3-8. The azimuth for each succeeding leg is then determined by adding the value of the measured angle at the occupied station to the value of the azimuth from the occupied station to the rear station. On occupation of each successive station, the first step is to compute the back azimuth of the preceding leg (the azimuth from the occupied station to the rear station). At the closing station, the azimuth carried forward is compared with the computed azimuth from the closing station to the reference azimuth mark.
10.2.1 AZIMUTH CORRECTION. The azimuth closure error is obtained by subtracting the known closing azimuth from the computed closing azimuth, as described above. This difference provides the angular closure error with the appropriate sign. By reversing this sign, the azimuth correction (with the appropriate sign) is obtained. If the angular error of closure is less than the allowable angular error of closure for the order of traverse (see closure standards in Chapter 4), the azimuths of the traverse may be adjusted. If the azimuth error is larger than the allowable closure error, then reobservations may be necessary. The allowable error of closure (or misclosure) depends on the instrument, the number of traverse stations, and the order of the control survey.

\[ \text{Allowable error of closure} = K \cdot \sqrt{n} \]  

(Eq 3-4)

where,

- \( K \) = fraction of the least count of the instrument, dependent on the number of repetitions and accuracy desired e.g., 20'' for Third-Order and 60'' for Fourth-Order-
- \( n \) = number of traverse stations

10.2.2 AZIMUTH ADJUSTMENT. The Compass Rule is based on the assumption that angular errors have accumulated gradually and systematically throughout the traverse. The angular correction is then distributed systematically (equally) among the angles in the traverse. Refer to the "Balanced Angle" column in the example at Figure 3-11 below where a 4-second misclosure was distributed equally.

10.2.3 TRAVERSE POSITION COMPUTATIONS. After the angles are adjusted as described above, compute the adjusted azimuth (or bearing) of each leg by using the starting azimuth and the adjusted angles at each traverse station. Verify the computed closing azimuth agrees with the computed fixed closing azimuth. Using the adjusted azimuths (or bearings) for each leg, and the measured distances (as corrected to sea level and grid scale), compute each traverse station X-Y (or N-E or departure-latitude) position from the beginning to the closing station--e.g., the "Unadjusted Latitudes and Departures" column in Figure 3-11. The linear misclosure at the closing station is determined in both X (departure or easting) and Y (latitude or northing) coordinates-- \( \Delta X \)
and ΔY. The overall position misclosure \( \sqrt{\Delta X^2 + \Delta Y^2} \) is then used to determine the relative accuracy (or precision) of the traverse, and conformance with the minimum closure standards in Table 4-1. The relative accuracy is obtained by dividing the misclosure (as computed after adjusting the angles) by the sum of the overall traverse length. This value is then inversed to obtain a ratio for comparison with Table 4-1, as shown in Equation 3-5 below.

\[
\text{Relative accuracy (or precision)} = \frac{\text{Misclosure (after angular adjustment)}}{\sum \text{of the traverse course distances}} \tag{Eq 3-5}
\]

The sample traverse shown in Figure 3-11 below resulted in a misclosure of 0.036 ft over full 603.1 ft traverse length. The relative accuracy is then:

\[
0.036 \div 603.1 \approx 1 / 17,000 \quad \text{(i.e., "1 part in 17,000" or 1:17,000)}
\]

The position misclosure (after azimuth adjustment) can then be distributed among the intermediate traverse station based on the adjustment rule being applied. For the Compass Rule, the latitude and departure misclosures are adjusted in proportion to the length of each traverse course divided by the overall traverse length. For any traverse leg with length \( dX \) (departure) and \( dY \) (latitude) in each coordinate, and with a final misclosure after azimuth adjustment of " \( \Delta X \) " and " \( \Delta Y \) " , the corrections to the \( dX \) or \( dY \) lengths are adjusted by:

\[
\text{Correction in } dX \text{ or } dY = \frac{- (\text{Misclosure } \Delta X \text{ or } \Delta Y) \cdot \text{(Length of Traverse Course)}}{\text{(Overall Traverse Length)}} \tag{Eq 3-6}
\]

Once the above corrections are applied to the latitudes and departures in each traverse course, the adjusted length and direction of each course can be computed, along with the final adjusted coordinates of each intermediate point. (These final computations are not shown in Figure 3-11).

10.2.4 ADJUSTMENT TECHNIQUES. In the past, the above adjustment was performed using a tabular form that was laid out to facilitate hand calculation of the angular and
coordinate corrections and adjustments—see sample at Figure 3-11 below. Today, COGO software packages can perform this adjustment in the field or office and these tabular computation forms are not necessarily needed.

![Figure 3-11](image)

Tabular computation format for a Compass Rule traverse adjustment
11. TRIANGULATION AND TRILATERATION SURVEYS. Triangulation and trilateration methods are now rarely used for expanding or densifying horizontal control. Before GPS, they were extensively used for this purpose. In USACE, localized triangulation and trilateration techniques (using Wild T3s and precise EDM) are still used for accurate structural deformation monitoring work. However, these specialized surveys are only performed around a lock, dam, or hydropower project.

11.1 GENERAL. A triangulation network consists of a series of angle measurements that form joined or overlapping triangles in which an occasional baseline distance is measured. The sides of the network are calculated from angles measured at the vertices of the triangle. A trilateration network consists of a series of distance measurements that form joined or overlapped triangles where all the sides of the triangles and only enough angles and directions to establish azimuth are determined.

11.2 NETWORKS. When practicable, all triangulation and trilateration networks should originate from and tie into existing coordinate control of equal or higher accuracy than the work to be performed. An exception to this would be when performing triangulation or trilateration across a river or some obstacle as part of a chained traverse. In this case, a local baseline should be set. Triangulation and trilateration surveys should have adequate redundancy and are usually adjusted using least squares methods.

11.3 ACCURACY. Point closure standards listed in Chapter 4 must be met for the appropriate accuracy classification to be achieved. If project requirements are higher-order, refer also to the FGCS "Standards and Specifications for Geodetic Control Networks" (FGCS 1984).

11.4 RESECTION. Three-point resection is a form of triangulation. Three-point resection may be used in areas where existing control points cannot be occupied or when the work does not warrant the time and cost of occupying each station. Triangulation of this type should be considered Fourth-Order, although Third-Order accuracy can be obtained if a strong triangular figure is used and the angles are
accurately measured. The following minimum guidelines should be followed when performing a three-point resection:

11.4.1 LOCATION. Points for observation should be selected to give strong geometric figures, such as with angles between 60 and 120 degrees of arc.

11.4.2 REDUNDANCY. If it is possible to sight more than three control points, the extra points should be included in the figure. If possible, occupy one of the control stations as a check on the computations and to increase the positioning accuracy. Occupation of a control station is especially important if it serves as a control of the bearing or direction of a line for a traverse that originates from this same point.

11.4.3 MEASUREMENTS. Both the interior and exterior angles should be observed and recorded. The sum of these angles should not vary by more than three (3) arc-seconds per angle from 360 degrees. Each angle should be turned not less than 2-4 times (in direct and inverted positions).