Chapter 3
Primary Control Surveys for Project Mapping

3-1. Purpose and Scope

Control surveys are performed to establish a monumented reference system for a military or civil facility mapping project. These fixed horizontal control points and vertical benchmarks are then used as starting points for supplemental topographic site plan mapping. This chapter provides guidance on the various techniques used to establish project control. FM 3-34.331 (*Topographic Surveying*) contains numerous examples of basic survey data reductions and computations, such as three-wire level reduction, level line reductions, c-factor computations, scale and grid factor computations, and traverse adjustments. This chapter will refer to the Field Manual for many of these topics.

a. 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 chapter will focus on current practices for performing "traditional" horizontal control surveys--i.e. control being established using total station traverse methods. (This chapter does contain some background on older survey methods for use in basic PROSPECT surveying courses). GPS control survey densification methods are more thoroughly covered in EM 1110-1-1003 (*NAVSTAR GPS Surveying*). Triangulation and trilateration methods will only be briefly addressed, along with references to other publications. Traverse survey methods described in this chapter are largely drawn from FM 3-34.331.

b. 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. Refer to EM 1110-1-1003 (*NAVSTAR GPS Surveying*) for details on performing accurate DGPS elevation transfers.

SECTION I
Traditional Horizontal Control Survey Techniques

3-2. General Overview

a. 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.

b. 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.

c. Monumentation. Primary project horizontal control points not permanently monumented in accordance with criteria and guidance established in EM 1110-1-1002 (*Survey Markers and Monumentation*) should meet the following minimum standards:

(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) 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.

(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.

(4) Sketches. A sketch should be placed in a standard field survey book or on a standard form, such as DA Form 1959 (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.

d. 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--see FGCS 1984.

COUNTRY Germany	TYPE OF MARK 170 Monument	STATION Stone Kamp	
LOCALITY Illesheim/L6528	STAMPING ON MARK NA	AGENCY (CAST IN MARKS) NA	ELEVATION 331.671 ~~(FT)~~ (M)
LATITUDE 49°28'10.47467"	LONGITUDE 10°23'10.92519"	DATUM WGS 84	DATUM Amsterdam
(NORTHING) (EASTING) 5,480,852.200 ~~(FT)~~ (M)	(EASTING) (NORTHING) 600,444.268 ~~(FT)~~ (M)	GRID AND ZONE 32U	ESTABLISHED BY (AGENCY) 320th Engineer
(NORTHING) (EASTING) (FT) (M)	(EASTING) (NORTHING) (FT) (M)	GRID AND ZONE	DATE (YYYYMMDD) 2001 07 15 / ORDER Third

TO OBTAIN GRID AZIMUTH, ADD ° ' " TO THE GEODETIC AZIMUTH
TO OBTAIN GRID AZ. (ADD) (SUB.) ° ' " TO THE GEODETIC AZIMUTH

OBJECT	AZIMUTH OR DIRECTION (GEODETIC)(GRID) (MAGNETIC)	BACK AZIMUTH	GEOD. DISTANCE (METERS) (FEET)	GRID. DISTANCE (METERS) (FEET)
	° ' "	° ' "		

The station is located on Storch Barracks, Illesheim, Germany.

To reach the station front gate of Storch 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.

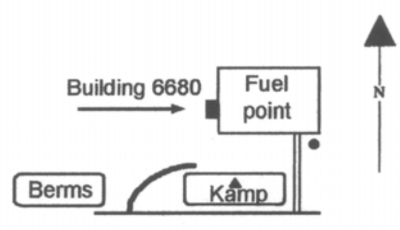
The station is a Type 70 monument protruding 20 cm above the ground and is located atop a burm.

The station is located 75.1 m at an azimuth of 160° from Building 6680, 82.3 m from the hot fuel point and 67 m from the fuel point sign.

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

Elevation was established by third-order leveling procedures.

SAMPLE



SKETCH

DA FORM 1959, JUL 2001 — REPLACES DA FORMS 1959 AND 1960, 1 FEB 57, WHICH ARE OBSOLETE.

DESCRIPTION OR RECOVERY OF HORIZONTAL CONTROL STATION
For use of this form, see FM 3-34.331; the proponent agency is TRADOC. USAPA V1.00

Figure 3-1. DA Form 1959 -- "Description or Recovery of Horizontal Control Station"

COUNTRY *USA*	TYPE OF MARK	STATION		
LOCALITY	STAMPING ON MARK	AGENCY (CAST IN MARKS) ***Corps of Engineers***	ELEVATION (FT) (M)	
LATITUDE	LONGITUDE	DATUM	DATUM	
(NORTHING) **(FT)** (M)	**(EASTING)** **(FT)** (M)	GRID AND ZONE	ESTABLISHED BY (AGENCY)	
(NORTHING) **(FT)** (M)	**(EASTING)** **(FT)** (M)	GRID AND ZONE	DATE	ORDER

TO OBTAIN	GRID AZIMUTH, ADD ° ' "	TO THE GEODETIC AZIMUTH
TO OBTAIN	GRID AZ. (ADD) (SUB) ° ' "	TO THE GEODETIC AZIMUTH

OBJECT	AZIMUTH OR DIRECTION (GEODETIC) (GRID) (MAGNETIC)	BACK AZIMUTH	GEOD DISTANCE (METERS) (FEET)	GRID DISTANCE (METERS) (FEET)
	° ' "	° ' "		

SKETCH

DA FORM 1959 JUL 2001 REPLACES DA FORMS 1959 AND 1960, 1FEB 57, WHICH ARE OBSOLETE.

DESCRIPTION OR RECOVERY OF HORIZONTAL CONTROL STATION
For use of this form, see FM 3-34.331; the proponent agnecy is TRADOC.

Figure 3-2. Blank DA Form 1959 -- "Description or Recovery of Horizontal Control Station"

e. 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.

f. 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-graduation distortions--see FM 3-34.331 for recommended circle settings for a 1" theodolite.

g. 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.

h. 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.

i. 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. These various calibrations and adjustments are described in the Appendix A-2 references or in FM 3-34.331 (*Topographic Surveying*).

j. 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), on DA Form 4253 (see FM 3-34.331), 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.
  - Temperature.
  - General atmospheric condition.
  - Wind.

Entry Page:

- Designation of the occupied station.
  - Full station name.
  - Year established.
  - 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.

k. Horizontal abstracts of directions. An abstract of horizontal directions should be compiled for every station at which horizontal directions have been observed. DA Form 1916 (see FM 3-34.331) 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 DA Form 1916 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.

⊼ @ FRANK, 1982							SEPT. 5, 1985 (5)
Object	D/R	Deg	Min	Sec	Mean	Diff"	P.C./Notes-Parker
Y JERRY	D	00	00	15			⊼- Long
1980	R	180	00	10	~~12.5~~		ϕ-Reet
							ϕ-Crider
CENTER,'85	D	95	48	39			
	R	275	48	33	36.0	23.5	95° 48′
PAT, '82	D	196	22	06			
	R	16	22	02	04.0	51.5	196° 21′
Y/JERRY'80	R	270	02	38			
	D	90	02	43	~~40.5~~		
CENTER,'85	R	15	50	56			
	D	185	50	59	57.5	17.0	95° 48′
PAT, '82	R	106	24	32			
	D	286	24	36	34.0	53.5	196° 21′

SAMPLE FIELD BOOK NOTES-THEODOLITE

Figure 3-3. Sample horizontal field book recording--Directional Theodolite

3-3. Secondary or Temporary Horizontal Control

a. 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.

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

(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.

(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) 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.

(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.

3-4. 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.

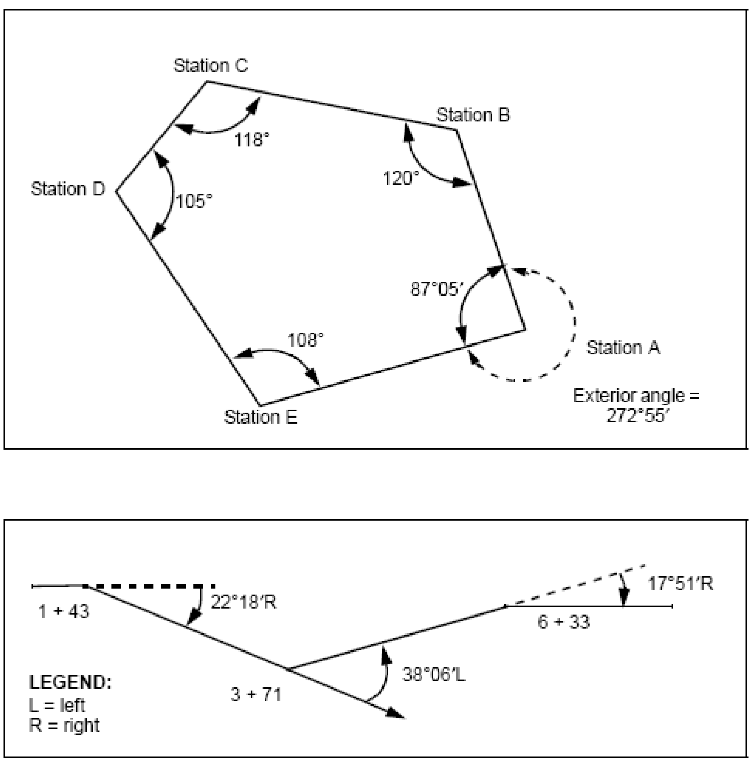


Figure 3-4. Interior angles on a closed traverse (top) and deflection angles on an open traverse (bottom) (FM 3-34.331)

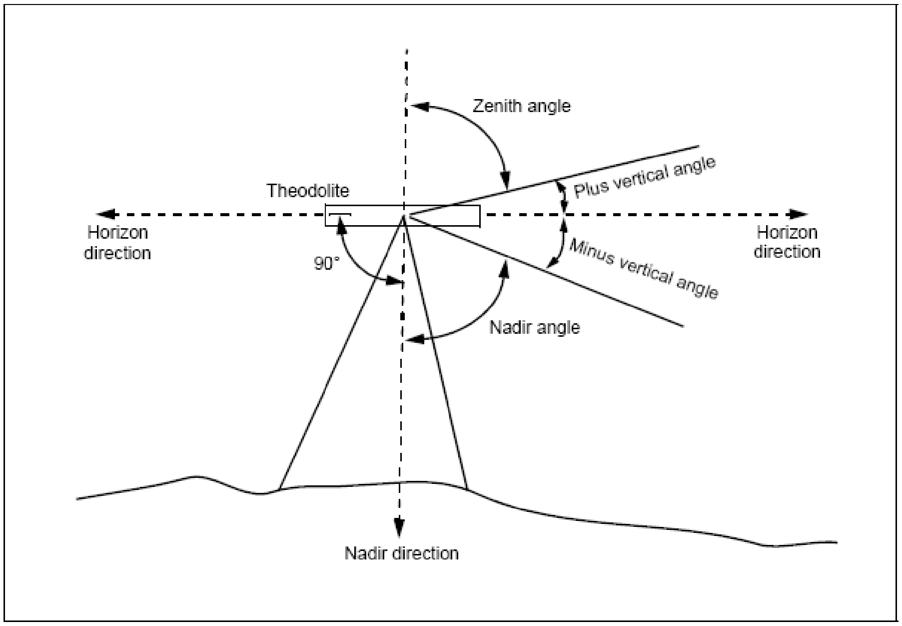


Figure 3-5. Reference directions for vertical angles--Horizontal, Zenith, and Nadir (FM 3-34.331)

a. 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.

b. 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.

c. 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.

d. 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.

e. 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).

3-5. 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.

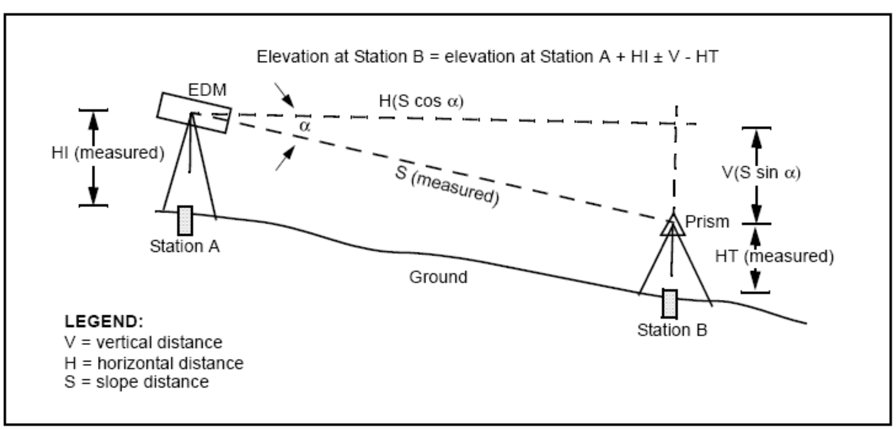


Figure 3-6. Geometry of an EDM measurement (FM 3-34.331)

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.

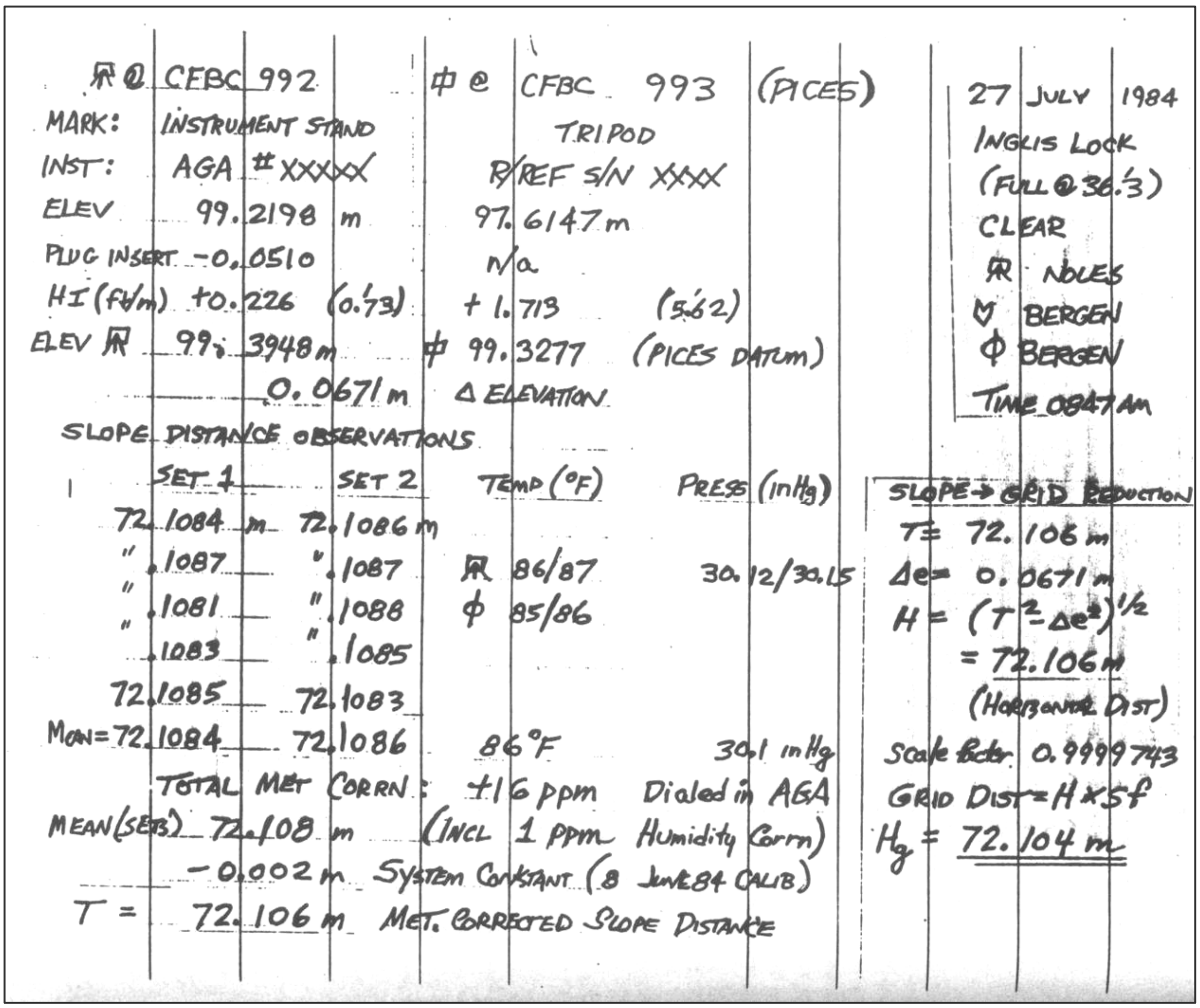

CFBC 992 @ CFBC 993 (PICES)
MARK: INSTRUMENT STAND TRIPOD
INST: AGA #XXXXX R/REF S/N XXXX
ELEV 99.2198 m 97.6147 m
PLUG INSERT -0.0510 n/a
HI (ft/m) +0.226 (0.73) +1.713 (5.62)
ELEV 99.3948 m 99.3277 (PICES DATUM)
0.0671 m Δ ELEVATION
SLOPE DISTANCE OBSERVATIONS
SET 1 SET 2 TEMP (°F) PRESS (inHg)
72.1084 m 72.1086 m
" .1087 " .1087 86/87 30.12/30.15
" .1081 " .1088 85/86
.1083 " .1085
72.1085 72.1083
MEAN=72.1084 72.1086 86°F 30.1 inHg
TOTAL MET CORRN: +16 ppm Dialed in AGA
MEAN (SETS) 72.108 m (INCL 1 PPM Humidity Corrn)
-0.002 m SYSTEM CONSTANT (8 JUNE 84 CALIB)
T = 72.106 m MET. CORRECTED SLOPE DISTANCE
27 JULY 1984
INGLIS LOCK
(FULL @ 36.3)
CLEAR
NOLES
BERGEN
BERGEN
TIME 0847 AM
SLOPE → GRID REDUCTION
T = 72.106 m
Δe = 0.0671 m
H = (T² - Δe²)^½
= 72.106 m
(HORIZONTAL DIST)
Scale Factor 0.9999743
GRID DIST = H x SF
Hg = 72.104 m

Figure 3-7. Horizontal distance observations and reductions--manual computations in field book (Jacksonville District)

a. 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.

b. 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.

3-6. 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.

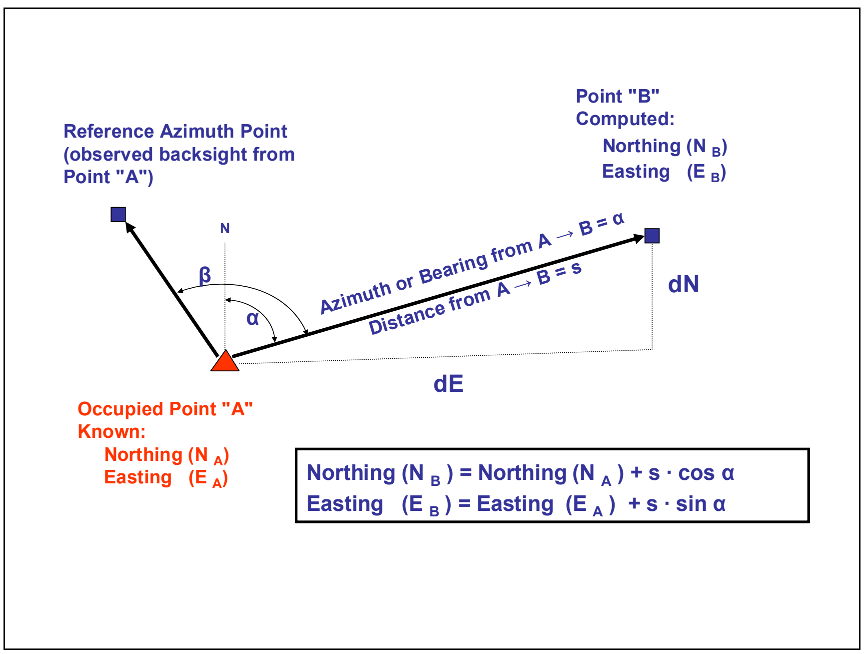


Figure 3-8. Forward Position Computation

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:

Azimuth (α) from A → B = 105° - (360° - 320°) = 65° [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:

$$dN = s \cdot \cos(\alpha)$$
$$dE = s \cdot \sin(\alpha) \qquad \text{(Eq 3-1)}$$

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.

dN = cos 70° 15" 15" · 568.78 = +0.337848 · 568.78 = +192.16 m

dE = sin 70° 15' 15" · 568.78 = +0.941200 · 568.78 = +535.34 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.

dN = cos 161° 12' 30" · 548.74 = -0.946696 ·548.74 = -519.49 m

dE = sin 161° 12' 30" · 548.74 = +0.322128 · 548.74 = +176.76 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.

dN = cos 294° 40' 45" · 783.74 = +0.417537 · 783.74 = +327.24 m

dE = sin 294° 40' 45" · 783.74 = -0.908660 · 783.74 = -712.15 m

3-7. 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.

a. Traverse types. There are two basic types of traverses, namely, closed traverses and open traverses.

(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.

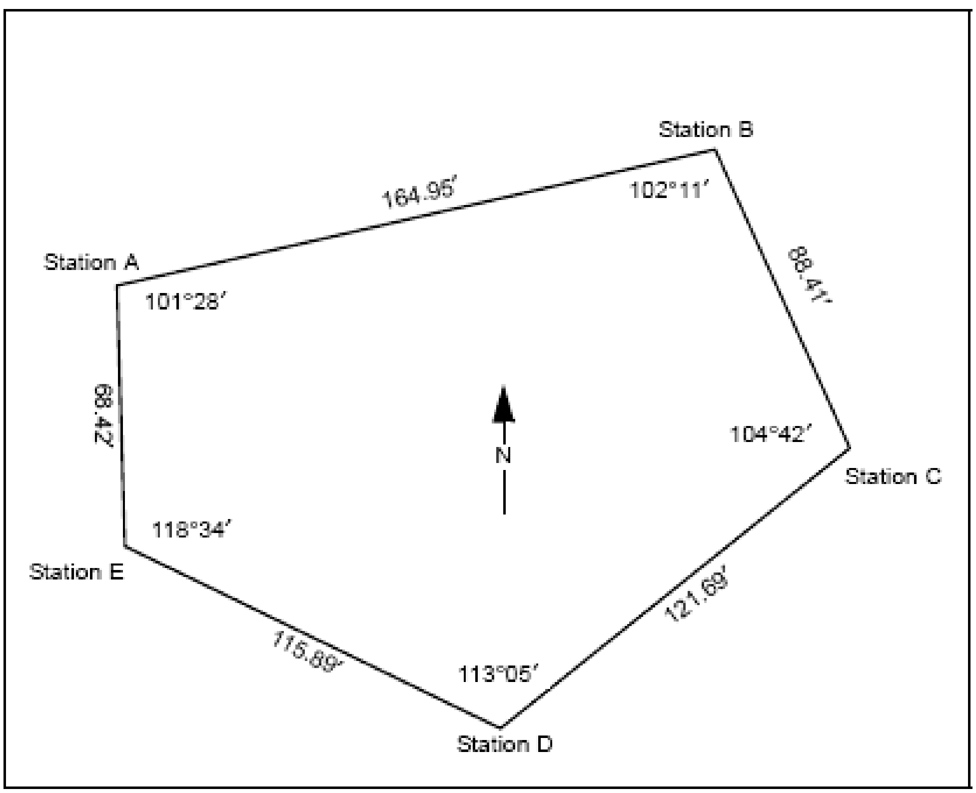


Figure 3-9. Closed Traverse--Looped (Station A fixed) or Connecting (Stations A and B fixed)

(a) 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) · 180º. If exterior angles were measured when performing a loop traverse, the true angular closure should equal (n+2) · 180º. 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.

(b) 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.

(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.

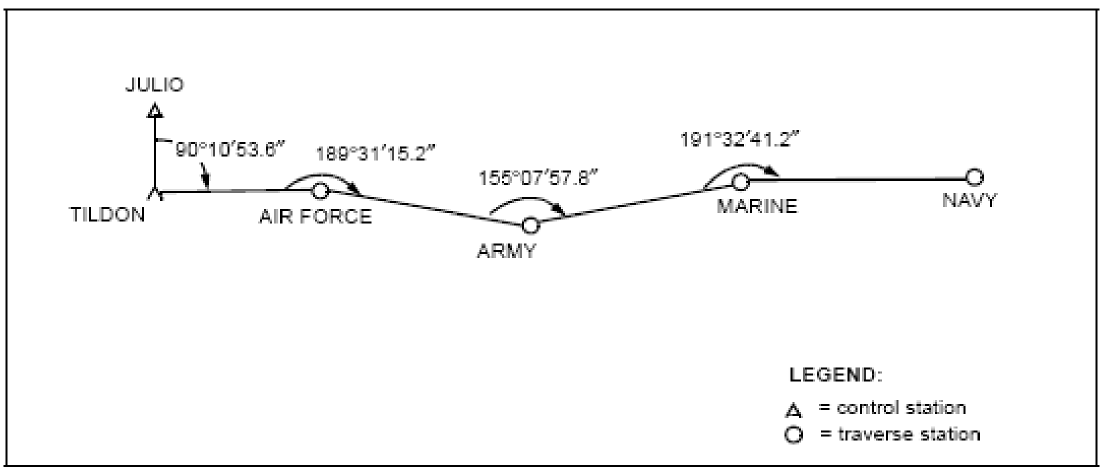


Figure 3-10. Open traverse

b. 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.

c. 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.

d. 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.

3-8. 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

The following general guidelines are recommended in performing traverse surveys:

a. 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. Refer to EM 1110-1-1002 (*Survey Markers and Monumentation*) for further guidance. 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.

b. 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 USACE 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.

c. 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.

d. 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.

e. 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.

f. 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.

g. 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. Refer to examples in FM 3-34.331. Temporary monuments need only be briefly described in the field notes

3-9. Traverse Computations and Adjustments

There are a number of methods available for adjusting traverses. The most common are listed below.

a. 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.

b. 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 540º 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.

c. 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.

$$v^t\, w\, v \rightarrow \text{minimum} \tag{Eq 3-2}$$

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.

$$l^\wedge = l + v \tag{Eq 3-3}$$

where
$l^\wedge$ = adjusted observation
l = observation
v = observation residual

(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 540º. If the correct model is not determined, the adjusted observations will be in error.

(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.

(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.

(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 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.

(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).

(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.

(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. Least squares adjustment techniques are covered in detail in EM 1110-1-1003 (*NAVSTAR Global Positioning System Surveying*) and EM 1110-2-1009 (*Structural Deformation Surveying*).

3-10. 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 any of the texts listed in Appendix A-2.

a. General. Traverse computations and adjustments require the following steps (Wolf and Brinker 1994):

- 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

b. 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.

(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.

$$\textit{Allowable error of closure} = K \cdot \sqrt{n} \qquad \text{(Eq 3-4)}$$

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--see Table 4-1 in Chapter 4)

n = number of traverse stations

(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.

c. 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-- ΔX 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.

$$\textit{Relative accuracy (or precision)} = \left[\textit{Misclosure (after angular adjustment)} \div \Sigma \textit{ of the traverse course distances}\right]^{-1} \qquad \text{(Eq 3-5)}$$

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

$0.036 \div 603.1 \approx 1 / 17{,}000$ (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 " ΔX " and " ΔY ", the corrections to the dX or dY lengths are adjusted by:

$$\text{Correction in dX or dY} = \frac{-\,(\text{Misclosure } \Delta X \text{ or } \Delta Y) \cdot (\text{Length of Traverse Course})}{(\text{Overall Traverse Length})}$$

(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).

d. 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. Other examples of Compass Rule traverse adjustments can be found in the references listed at Appendix A. Today, COGO software packages can perform this adjustment in the field or office and these tabular computation forms are not necessarily needed.

Compass Rule Adjustment

Station	Measured Angle	Balanced Angle	Azimuth	Horiz. Distance	Unadjusted Latitude	Unadjusted Depart	Adjusted Latitude	Adjusted Depart	Coordinates	Adjusted Length	Adjusted Direction
12			103°-03'-14"	110.84'							
11	85°-05'-33"	85°-05'-34"	188°-03'-48"	219.51'	-217.29'	-31.11'	-217.30'	-31.12'			
9'	58°-48'-39"	58°-48'-40"	66°-57'-28"	130.05'	50.90'	119.67'	50.90'	119.66'			
13'	120°-52'-29"	120°-52'-30"	7°-49'-58"	142.70'	141.37'	19.45'	141.36'	19.44'			
12	95°-13'-15"	95°-13'-16"	288°-03'-14"	110.84'	25.04'	-107.98'	25.04'	-107.98			
		Σ = 360°									
$\Sigma\alpha_m$ =	359°-59'-56"										
α_t =	360°										
α_e =	-4"										
			D =	Σl_{ij}	$\Sigma \Delta N_m$	$\Sigma \Delta E_m$	$\Sigma \Delta N$	$\Sigma \Delta e$			
			SUM	603.10'	0.02'	0.03'	0	0			

(ρ) Line of closure = 0.036' Area = 20,081 Square Feet

(P) Precision = 1/16,726 Area = 0.46 Acres

Adjustment by BAF Rule COMPASS RULE

(ρ) Line of closure $\left(\rho = \sqrt{\sum N_m^2 + \sum E_m^2}\right)$

(P) Traverse precision $\left(\frac{P}{D}\right)$

Figure 3-11. Tabular computation format for a Compass Rule traverse adjustment

3-11. 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. Procedures and techniques for deformation triangulation/trilateration surveys are found in EM 1110-2-1009 (*Structural Deformation Surveying*).

a. 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.

b. 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.

c. 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).

d. 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:

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

(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.

(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).

SECTION II
Vertical Control Survey Techniques

3-12. General

Vertical control surveys provide a basic framework for controlling elevations on facility mapping projects. The purpose of vertical control surveys is to establish elevations on rigid benchmarks throughout the project area. These benchmarks can then serve as points of departure and closure for leveling operations and as reference benchmarks during subsequent construction work. The NGS, USGS, other Federal agencies, and many USACE commands have established vertical control throughout the CONUS. Unless otherwise directed, these benchmarks should be used as a basis for all vertical control surveys. Descriptions of benchmark data and their published elevation values can be found in data holdings issued by the agency maintaining the project/installation. Information on USACE maintained points can be found at District or Division offices. This section focuses on Second-Order and Third-Order vertical control techniques performed using differential leveling instruments--Figure 3-12 below.

Figure 3-12. Sokkia B20 automatic level (Key West Harbor Dredging Project 2004-- C& C Technologies, Inc. & Jacksonville District)

a. Differential leveling. With differential leveling, differences in elevation are measured with respect to a horizontal line of sight established by the leveling instrument. Once the instrument is leveled (using either a spirit bubble or automated compensator), its line of sight lies in a horizontal plane. Leveling comprises a determination of the difference in height between a known elevation and the instrument and the difference in height from the instrument to an unknown point by measuring the vertical distance with a precise or semi-precise level and leveling rods (Figure 3-13). Digital (or Bar Code) levels are used to automatically measure, store, and compute heights, and are capable of achieving Second-Order or higher accuracies. Accuracy standards should follow the point closure standards shown in Table 4-2 in Chapter 4. When leveling in remote areas where the density of basic vertical control is scarce, the semi-precise rod is generally used. The semi-precise rod should be graduated on the face to centimeters and the back to half-foot intervals. When leveling in urban areas or areas with a high density of vertical control where ties to higher-order control are readily available, the standard leveling rods are used--e.g., a Philadelphia rod graduated to hundredths of a foot. Other rods that are graduated to centimeters can be used. Both types of rods are furnished with targets and verniers that will permit reading of the scale to millimeters or thousandths of a foot if required by specifications. This is generally not required on lower-order level lines. Standard stadia rods may also be used for lower-order level lines. The stadia rod is graduated to the nearest 0.05 ft, or two centimeters. These rods are generally equipped with targets or verniers, but if project specifications require, they can be estimated to hundredths of a foot.

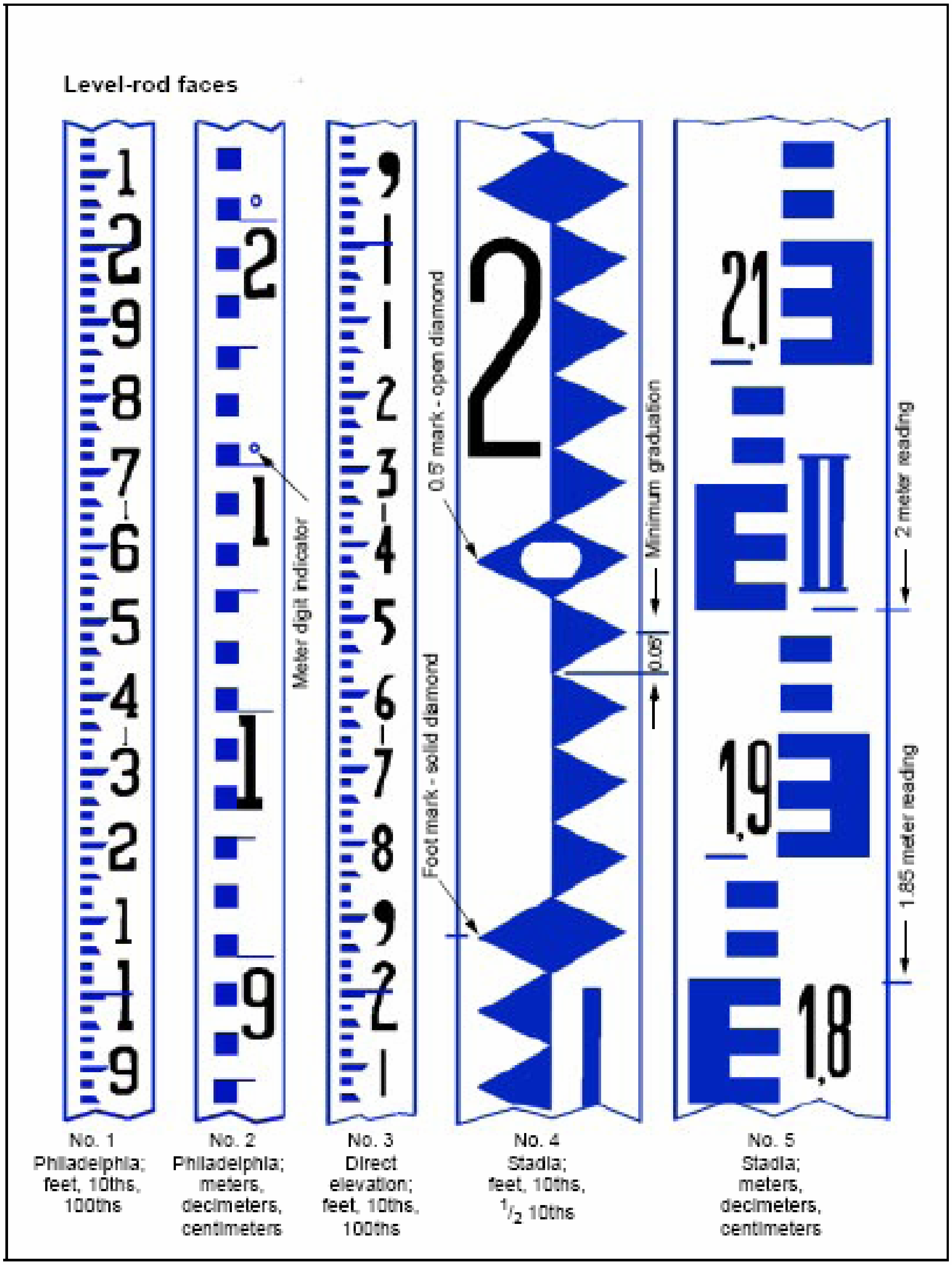


Figure 3-13. Traditional rectangular cross-section leveling rods showing a variety of graduation markings (FM 3-34.331)

b. Trigonometric leveling. This method applies the fundamentals of trigonometry to determine the differences in elevation between two points by observing a horizontal distance and the vertical angles above or below a horizontal plane. Trigonometric leveling is generally used for lower-order accuracy vertical positioning; however, it is sufficiently accurate for radial topography when elevations of features are cut in by a total station. Trigonometric leveling is especially effective in establishing control for profile lines, for strip photography, and in areas where the landscape is steep. With trigonometric leveling operations, it is necessary to measure the height of instrument (HI) and rod target above the monuments, the slope distance (s), the vertical angle (a), and the rod intercept. From this data, the vertical difference in elevation (DE) can be computed using the sine of the vertical angle and applying the rod difference (Figure 3-14 below). Refinements to this technique include doubling vertical angles, taking differences from both stations, and using the mean values. If the horizontal distance is known between the instrument and the rod, it is not necessary to determine the slope distance. The instrument most commonly used for trigonometric leveling is a directional theodolite or Total Station. Manufacturer specifications and procedures should be followed to achieve the desired point closure standards.

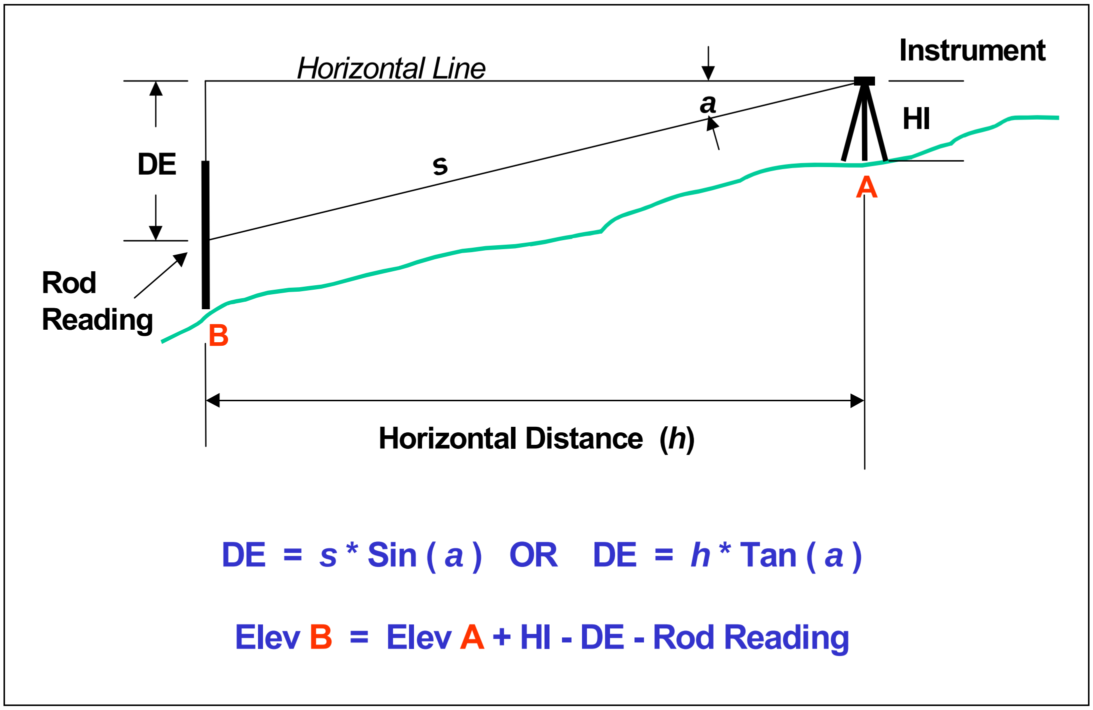


Figure 3-14. Trigonometric leveling

(1) Recording vertical trigonometric observations (zenith distances--ZD) is the same for all orders of accuracy. Vertical observations may be recorded in a standard field book, on DA Form 5817-R (*Zenith Distance/Vertical Angle*--see FM 3-34.331), on an equivalent single-sheet recording form, or a data collector. In all cases, complete documentation will be performed in the field. The following information is typically required:

- The HI above the station (recorded to the nearest 0.01 meter or foot).
- A sketch of the observed target (that shows the point observed on the target) at the bottom of the object-observed column.
- The height of the observed rod reading (height of target--HT) above the station being observed (recorded to the nearest 0.01 meter or foot).
- A sketch showing any target's adjoining stations. This sketch should be drawn in the bottom of the remarks column. All possible points that may be observed should be measured and recorded to the nearest 0.01 meter or foot.

(2) During vertical observations, the time of the first observation of the first position and the time of the last observation of the last position are recorded. The times are recorded to the nearest whole minute.

(3) Vertical observations may alternatively be abstracted onto DA Form 1943 (*Abstract of Zenith Distances*--see FM 3-34.331) at the station site by the observing party. Vertical observations recorded as vertical angles are converted to ZDs before abstracting. Targets or signals shown to other stations are sketched and dimensioned at the bottom of the form. If a target or signal is changed during the day, the time of the change and the new dimensions are also entered.

c. Trigonometric elevations over longer lines. Trigonometric elevations over longer lines may need to be corrected for curvature and refraction. These corrections are insignificant (< 0.02 ft) and unnecessary for topographic survey distances of 1,000 feet or less. The type of correction used depends on whether the long line was occupied at each end. The following formula (Wolf and Brinker 1994) is used to determine the combined curvature and refraction correction for trigonometric elevations observed over longer lines.

$$h\ (feet) = 0.0206\ (F)^2 \tag{Eq 3-7}$$

where

h = *combined correction for curvature and refraction in feet*
F = *length of observed line in thousands of feet*

As an example, given a 2,000 ft line, the combined correction would be 0.0206 (2) 2 = 0.08 ft, or about 0.1 ft. For longer lines, these approximate computations are not accurate; however, there are few applications requiring trigonometric leveling over longer lines given GPS methods will yield more accurate results. If long-line trigonometric leveling is required, consult the NOAA/NGS for more accurate observing procedures and computations.

d. Barometric leveling. This method uses the differences in atmospheric pressure as observed with a barometer or altimeter to determine the differences in elevation between points. This method is the least accurate of determining elevations. Because of the lower achievable accuracies, this method should only be used when other methods are not feasible or would involve great expense. Generally, this method is used for elevations when the map scale is to be 1:250,000 or smaller.

e. Reciprocal leveling (Valley or River Crossings). Reciprocal leveling is a method of carrying a level circuit across an area over which it is impossible to run regular differential levels with balanced sights (Figure 3-15). Most level operations require a line of sight to be less than 300 or 400 feet long. However, it may be necessary to shoot 500-1,000 feet, or even further, in order to span across a river, canyon, or other obstacle. Where such spans must be traversed, reciprocal leveling is appropriate. The reciprocal leveling procedure can be described as follows. Assume points "A" and "B" are turns on opposite sides of the obstacle to be spanned (Figure 3-15) where points A and B are intervisible. Two calibrated rods are used, one at point A, and the other at point B. With the instrument near A, read rod at A, then turn to B and have target set as close as possible and determine the difference in elevation. Leaving rods at A and B, move the instrument around to point B, read B, then turn to read A and again determine the difference in elevation. The mean of the two results is the final height difference to be applied to the elevation of A to get an elevation value for point B. If the long sight is difficult to determine, it is suggested that a target be used and the observations repeated several times to determine an average value. For more precise results it will be necessary to take several foresights, depending on the length of the sight. It is typical to take as many as 20 to 30 sightings. When taking this many sightings, it is critical to relevel the instrument and reset the target after each observation. Reciprocal leveling assumes the conditions during the survey do not change significantly for the two positions of the level. Reciprocal leveling with two instruments should never be done unless both instruments are used on both sides of the obstacle and the mean result of both sets used. The use of two instruments is advised if it is a long trip around the obstacle. Reciprocal leveling is effective only if the instruments used will yield measurements of similar precision.

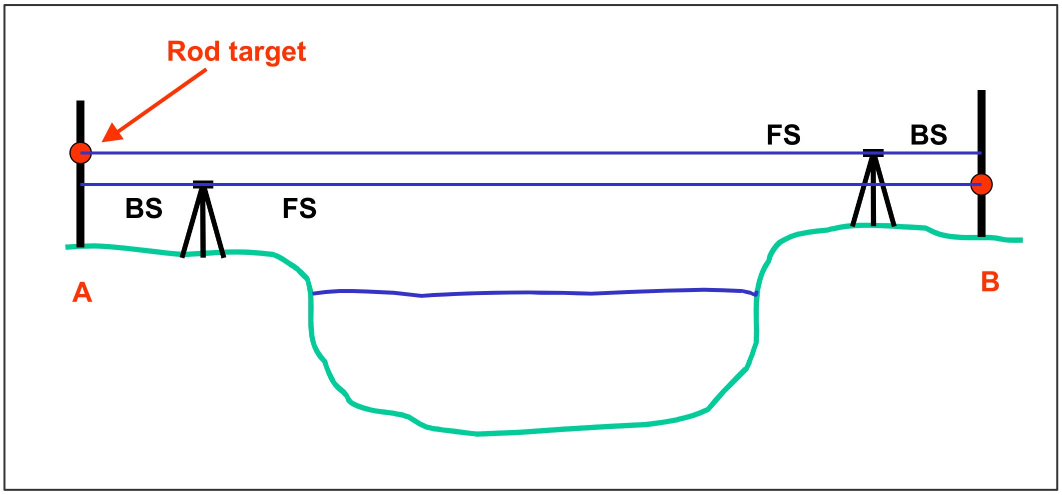


Figure 3-15. Reciprocal leveling for river crossing

f. Two rod leveling. In order to increase the productivity in precise leveling operations, it is advisable to use two rods. When the observations are completed at any instrument setup, the rods and the instruments are moved forward simultaneously. An even number of setups should be used to minimize the possible effects of rod index error. Two rods are recommended when using an automatic level, as this takes full advantage of the productivity possible with this type of instrument.

g. Tidal benchmarks and datums. For guidance on the establishment of tidal benchmarks and datums refer to Appendix B (*Requirements and Procedures for Referencing Coastal Navigation Projects to Mean Lower Low Water (MLLW) Datum*) and EM 1110-2-1003 (*Hydrographic Surveying*).

3-13. Second-Order Leveling

a. General. As shown in Figure 3-16 below, a leveling operation consists of holding a rod vertically on a point of known elevation. A level reading is then made through the telescope to the rod, known as a backsight (BS), which gives the vertical distance from the ground elevation to the line of sight. By adding this backsight reading to the known elevation, the line of sight elevation, called "height of instrument" (HI), is determined. Another rod is place on a point of unknown elevation, and a foresight (FS) reading is taken. By subtracting the FS reading from the height of instrument, the elevation of the new point is established. After the foresight is completed, the rod remains on that point and the instrument and back rod are moved to forward positions. The instrument is set up approximately midway between the old and new rod positions. The new sighting on the back rod is a backsight for a new HI, and the sighting on the front rod is a FS for a new elevation. The points on which the rods are held for foresights and backsights are called "turning points." Other foresights made to points not along the main line are known as "sideshots." This procedure is used as many times as necessary to transfer a point of known elevation to another distant point of unknown elevation.

STA	BS (+)	HI		FS (-)	ELEV				DESCRIPTION
BM Hood-2	6.35	630.22			623.87				Bronze disk ...
TP 1				1.96	628.26				
TP 2									
TP 3									
TP 4									

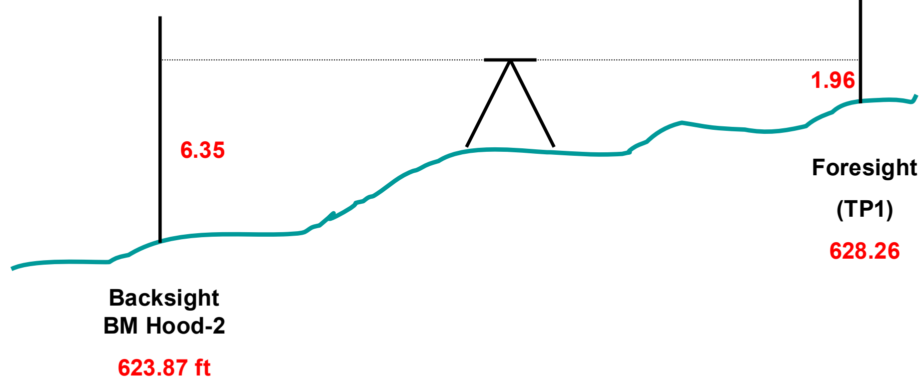


Figure 3-16. Differential leveling--example of one setup between benchmark and TP 1 (Standard field book recording format shown at top of sketch)

b. Leveling accuracy. Second-Order leveling point closure standards for vertical control surveys are shown in Table 4-2. Second-Order leveling consists of lines run in only one direction, and between benchmarks previously established by First-Order methods. If not checking into another line, the return for Second-Order--Class I level work should check within the limits of 0.025 times the square root of M feet (where "M" is the length of the level line in miles), while for Second-Order Class II work, it should check within the limits of 0.035 times the square root of M feet. Ties to two or more benchmarks are always recommended in order to verify stability of the fixed benchmarks.

c. Leveling equipment. The type of equipment needed is dependent on the accuracy requirements. Examples of precise leveling instruments are shown in Figure 3-17 below.

(1) Second-Order leveling instrument. Second-Order leveling instruments require a relatively precise level. Often a graduated parallel plate micrometer is built into the instrument to allow reading to the nearest 0.001 of a unit. The sensitivity of the level vial, telescopic power, focusing distance, and size of the objective lens are factors in determining the precision of the instrument. Instruments are rated and tested according to their ability to maintain the specified order of accuracy. Only those rated as precise geodetic quality instruments may be used for Second-Order work.

(2) Precise level rods. Precise level rods are normally used when running Second-Order levels. Both traditional rods and bar code type rods may be used. The rods may be of one piece, invar strip type, with the least graduation on the invar strip of 1 centimeter. The front of the rod is graduated in meters, decimeters and centimeters on the invar strip. The back of the rod is graduated in feet and tenths of feet, or yards and tenths of yards. Rods with similar characteristics are paired and marked. The pairings must be maintained throughout a line of levels. The invar strips should be checked periodically against a

standard to determine any changes that may affect their accuracy. The precise level rod is a scientific instrument and must be treated as such; not only during use but also during storage and transporting. When not in use they must be stored in their shipping containers to avoid damage. The footpiece should be inspected frequently to make sure it has not been bent or otherwise damaged.

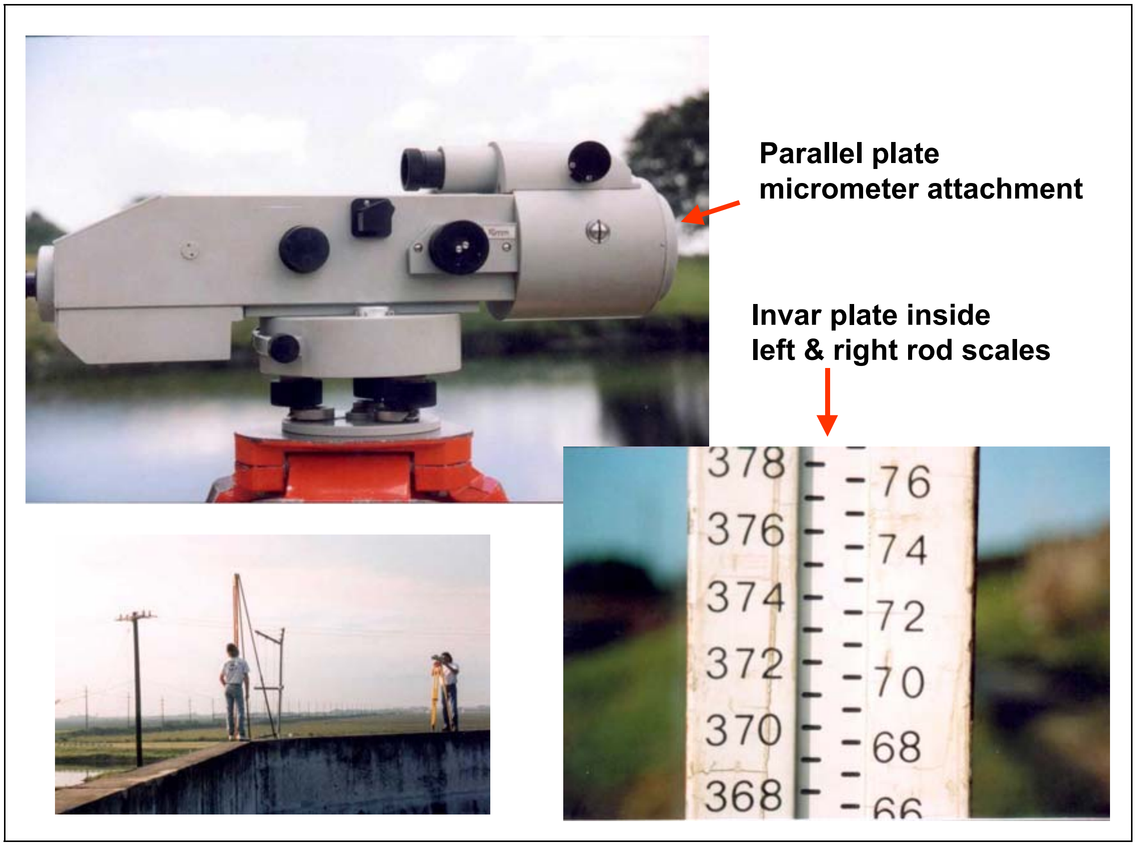


Figure 3-17. Zeiss Ni1 automatic level with parallel plate micrometer attached--precise double-scale Invar rod with constant 3.01550-meter difference in left and right scales (Jacksonville District)

d. Leveling monumentation. All benchmarks used to monument Second-Order level lines should conform to criteria published in EM 1110-1-1002 (*Survey Markers and Monumentation*). Benchmarks used to monument Second-Order level lines should be standard USACE brass caps set in concrete. The concrete should be placed in holes deep enough to avoid local disturbance. If the brass cap is not attached to an iron pipe, use some type of metal to reinforce the concrete prior to embedding the brass cap. Concrete should be placed in a protected position. If possible, benchmarks should be set close to a fence line, yet far enough away to permit plumbing of level rod. Do not set monuments closer than four feet to a fence post, as the benchmark likely will be disturbed if the post is replaced. Each brass cap must be stamped to identify it by the methods detailed in EM 1110-1-1002. In addition to stamping a local number or name on the cap, it is optional to stamp the elevation on the brass cap after final elevation adjustment has been made. The benchmarks must be set no less than 24 hours in advance of the level crew if the survey is to be made immediately after monument construction.

e. Leveling notes. Notes for Second-Order levels should be kept in a format approved by the District, or should follow recognized industry practice. A set style cannot be developed due to different types of equipment that may be employed. Elevations generally should not be carried in the field as they will be adjusted by the field office and closures approved prior to assigning a final adjusted elevation. See the following section on Third-Order leveling for sample recording formats and sketches.

f. Three-wire leveling. This method can be used for most types of leveling work and will achieve any practical level of accuracy, including Second-Order. However, most applications do not require the accuracies possible with three-wire leveling; plus, it is somewhat labor intensive. Three-wire leveling can be applied if the reticule of the level has stadia lines and substadia that are spaced so that the stadia intercept is 0.3 ft at 100 feet, rather than the more typical 1.0 ft at 100 feet. The substadia lines in instruments meant for three-wire leveling are short cross lines that cannot be mistaken for the long central line used for ordinary leveling. Although there are many different observing techniques for three-wire leveling, in the following example, the rod is read at each of the three lines and the average is used for the final result. Before each reading, the level bubble is centered. The half-stadia intervals are compared to check for blunders. The following values were taken and recorded and calculations made:

Upper Wire:	8.698	2.155 :Upper Interval
Middle Wire:	6.543	
Lower Wire:	4.392	2.151 :Lower Interval
Sum	19.633	
Average	6.544	

The final rod reading would be 6.544 feet. The upper and lower intercepts differ by only 0.004 ft--an acceptable error for this sort of leveling and evidence that no blunder has been made. It is recommended that "Yard Rods" specifically designed for three-wire leveling operations be used instead of Philadelphia rods that are designed for ordinary leveling. A sample recording form is shown at Figure 3-18.

THREE-WIRE LEVELING
For use of this form, see FM 3-34.331; the proponent agency is TRADOC.

INST. OP. INT. | 1st COMP. INT. | 2nd COMP. INT.

PROJECT: Example | LOCATION: Fort Belvoir, Virginia | ORGANIZATION: DMS
OBSERVER: SFC Jones | RECORDER: SGT Smith | INSTRUMENT: Wild NA2 - 1234 | SUN: Warm | WIND: Windy | WEATHER: Clear
FROM: D2 | TO: BASS | DATE (YYYYMMDD): 2001 07 15 | TIME: 0830-0920 | LINE OR NET: Training 1 | PAGE NO.: no. 2 | NO. OF PAGES: 4

STATION	BACKSIGHT FACE OF ROD	MEAN	BACK OF ROD	INTERVAL	SUM OF INTERVALS	FORESIGHT FACE OF ROD	MEAN	BACK OF ROD	INTERVAL	SUM OF INTERVALS	REMARKS
D2	1201					2963					
	0850	0850.3		351		2623	2622.3		340		
	0500			350	701	2281			342	682	
	2551	0850.3			701	7867	2622.3			682	
	2657					0899					
	2406	2406.0		251		0638	0637.7		261		
	2155			251	502	0376			262	523	
	9769	3256.3			1203	9780	3260.0			1205	
	3081					1361					
	2779	2779.3		302		1050	[illegible]		311		
	2478			301	603	[illegible]			311	622	
	18107	+6035.6			[illegible]	[illegible]	[illegible]310.0			1827	BASS
		-4310.0								1806	
	BDE =	+1.7256							B distance	3633	
									km	0.3633	
FDE =	-1.7276				AE = 0.012	√distance km		F distance	0.3302		
BDE =	+1.7256				AE = ±0.012	√0.3302					
								B distance	0.3633		
EC =	-0.0020				AE =	±0.0068					

DA FORM 5820, JUL 2001 — EDITION OF AUG 89 IS OBSOLETE. — USAPA V1.00

Figure 3-18. Three-wire leveling recording form--DA Form 5820 (FM 3-34.331)

g. High accuracy differential leveling. Methods and procedures for conducting highly precise (i.e. First-Order) differential leveling used for monitoring structural settlements are covered in EM 1110-2-1009 (*Structural Deformation Surveying*).

3-14. Third-Order and Lower-Order Leveling

a. General. Leveling run for traverse profiles, temporary benchmarks, control of cross-sections, slope stakes, soundings, topographic mapping, structure layout, miscellaneous construction layout, and construction staking should be Third- or Fourth-Order leveling, unless otherwise directed. All levels should originate from and tie into existing control; preferably from two or more benchmarks. No level line should be left unconnected to control unless by specific instructions of the survey supervisor or written directive.

b. Leveling accuracy. All accuracy requirements for USACE vertical control surveys will conform to the point closure standards shown in Table 4-2. The required accuracy (in feet) for Third-Order levels is 0.050 times the square root of M feet, and where "M" is the length of the level line in miles. Construction Layout level work will conform to 0.100 times the square root of M feet. The length of the line may be determined from quad sheets or larger scale map if a direct measure between points is not available.

c. Leveling equipment. The type of equipment needed is dependent upon the accuracy requirements.

(1) Third-Order level. A semi-precise level can be used for Third-Order leveling, such as the tilting Dumpy type, three-wire reticule, or equivalent.

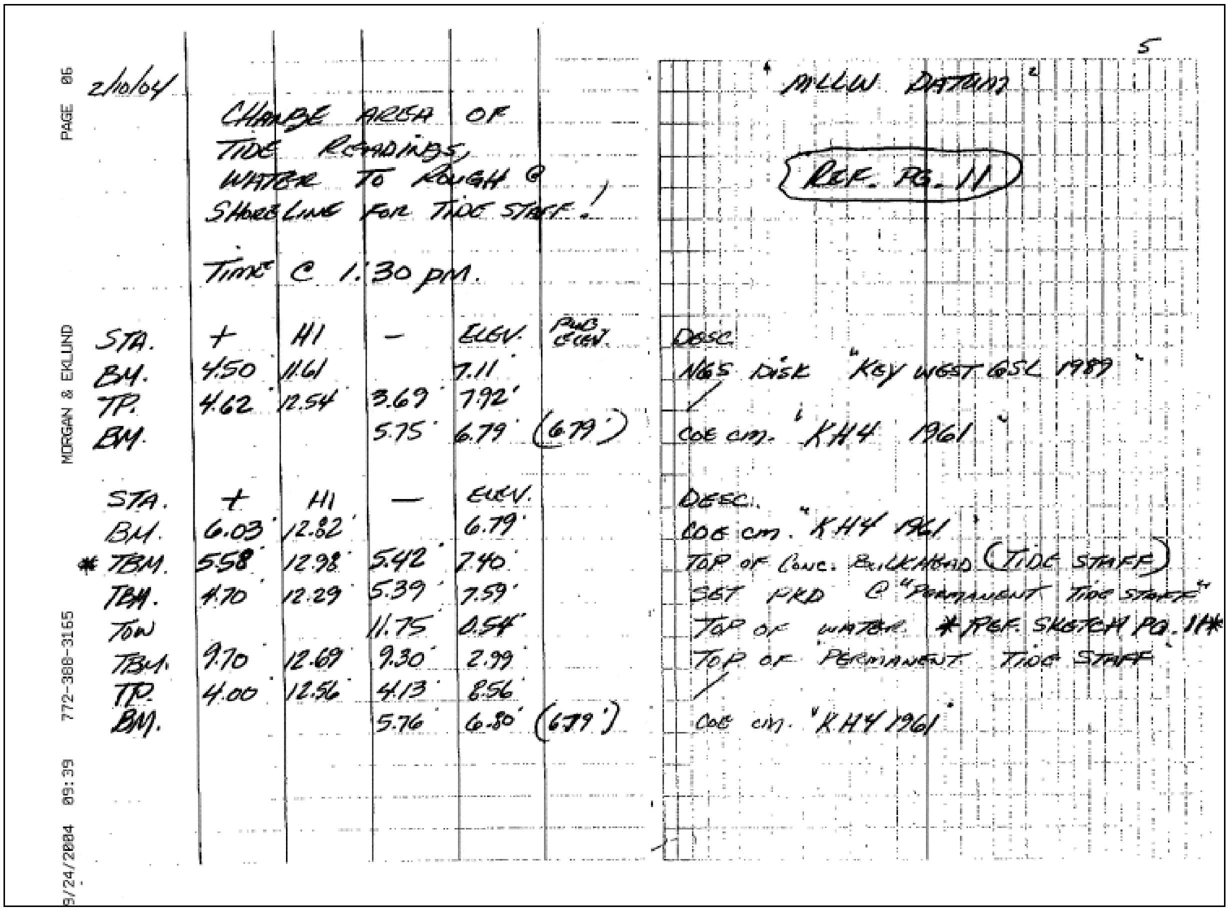
(2) Leveling rods. The rods should be graduated in feet, tenths and hundreds of feet. The Philadelphia rod or its equivalent is acceptable. However, the project specifications will sometimes require that semi-precise rods be used that are graduated on the front in centimeters and on the back in half-foot intervals. The Zeiss stadia rod, fold type, or its equivalent should be used when the specifications require semi-precise rods.

(3) Lower-order instruments. The type of spirit level instrument used should ensure accuracy in keeping with required control point accuracy. Precision levels are not required on lower-order leveling work. The Fennel tilting level, dumpy level, Wye level, or their equivalent, are examples of levels that can be used. A stadia rod with least readings of 0.05 ft or 1 cm will be satisfactory. The use of turning pins and/or plates will depend upon the type of terrain or if rods may be placed on firm stones or roadways.

d. Leveling monumentation. The level line should be tied to all existing benchmarks along or adjacent to the line section being run. In the event there are no existing benchmarks near the survey, new ones should be set, not more than 0.5 mile apart. Steep landscape in the area of survey may require monuments to be set at a closer spacing. Benchmarks should be set on permanent structures, such as, head walls, bridge abutments, pipes, etc. Large spikes driven into the base of trees, telephone poles, and fence posts are acceptable for this level of work. All temporary benchmarks must have a full description including location. Unless they are on a turn, they are not considered temporary benchmarks. No closures shown by an intermediate shot will be accepted. All temporary benchmarks must have a name or number for future identification.

e. Leveling equipment. Turning pins should be driven into in the ground until rigid with no possibility of movement. Turning points or temporary benchmarks will have a definite high point so that any person not familiar with the point will automatically hold the rod on the highest point, and so that it can spin free. If solid rocks are being used for turns they must be marked with crayon or paint prior to taking readings. It is not mandatory to use targets on the rod when the reading is clearly visible. However, they are required in dense brush, when using grade rods, or when unusually long shots are necessary. Rod bubbles should always be used to ensure the rod is held plumb.

f. Leveling notes and sketches. Complete notations or sketches should be made to identify level lines and side shots. All Second- and Third-Order or lower level notes should be completely reduced in the field as the levels are run; with the error of closure noted at all tie in points. In practice, the circuit will be corrected to true at each tie in point unless instructed to do otherwise by the survey supervisor or written directive. Any change in rod reading should be initialed and dated so there is no doubt as to when a correction was made. Cross out erroneous readings--never erase them. The instrument man should take care to keep peg notes on all turns in the standard field book. The notes should be dated and noted as to what line is being run, station occupied, identification of turns, etc. A complete description of each point on which an elevation is established should be recorded in the field book adjacent to the station designation. Entries should be made in the book that give the references to the traverse notes and other existing data used for elevations (e.g., TRAVERSE BOOK XXXX PAGE XX, USGS Quad XXXXXX, NOS Chart XXXX, etc.). Level notes should conform to a standard industry format, e.g., Point--(+)BS--HI--(-)FS--Elev. In general, level notes should follow the formats shown in Figures 3-19 through 3-22 below. However, local variations are acceptable--see Kavanagh 1997 for examples of different types of level notes. Level line sketches should be drawn in the field, as shown in the example at Figure 3-23 below. These sketches are particularly helpful in resolving complicated loop closures or where redundant lines have been run. Sketches may be drawn in a field book or on graph paper. They need not be at any particular orientation or scale. Original field notes or copies of notes may be submitted, as directed.


PAGE 05 MORGAN & EKLUND 772-388-3165 09:39 9/24/2004

5

2/10/04

CHANGE AREA OF TIDE READINGS, WATER TO ROUGH @ SHORELINE FOR TIDE STAFF!

TIME @ 1:30 PM.

MLLW DATUM

REF. PG. 11

STA.	+	HI	-	ELEV.	PUB ELEV.	DESC.
BM.	4.50	11.61		7.11		NGS DISK "KEY WEST GSL 1989"
TP.	4.62	12.54	3.69	7.92		
BM.			5.75	6.79	(6.79)	COE CM. "KH4 1961"

STA.	+	HI	-	ELEV.		DESC.
BM.	6.03	12.82		6.79		COE CM. "KH4 1961"
* TBM.	5.58	12.98	5.42	7.40		TOP OF CONC. BULKHEAD (TIDE STAFF)
TBM.	4.70	12.29	5.39	7.59		SET PKD @ "PERMANENT TIDE STAFF"
TOW			11.75	0.54		TOP OF WATER *REF. SKETCH PG. 11*
TBM.	9.70	12.69	9.30	2.99		TOP OF PERMANENT TIDE STAFF
TP.	4.00	12.56	4.13	8.56		
BM.			5.76	6.80	(6.79)	COE CM. "KH4 1961"

Figure 3-19. Sample single-wire level notes to set tide staff (Jacksonville District--Morgan & Ekland, Inc.)

LEVELS TO BLDG. AREA - TOPO CONTROL — 9-1-75 (5)

STA.	+		−	UNADJ. ELEV.	ELEV.		
MON "A"	~~2.20~~ 1.90				278.47	USCE-1974	Clear, warm; ⊼ ∇ HALL; ɸ HILL
			~~10.43~~ ~~10.02~~				
	0.28						
	0.10		12.39				
	0.09		11.46				
	7.49		6.52				
	2.65		5.26				
	9.95		0.59				
	10.25		0.44				
	4.13		4.22				
TBM# 1	36.54	−19.39	4.62	259.08	+.02 259.10	9 turns	TBM#1 is a chiseled □ in the NE headwall of culvert.
			55.93				
	0.93						
	1.84		11.05				
	1.27		3.28				
	2.57		10.07			4 turns	
MON "B"	6.61	−21.41	3.62	237.67	+.03 237.70	USCE 1974	SAMPLE NOTES LEVELS
			28.02				

SAMPLE FIELD BOOK NOTES: LEVELS

Figure 3-20. Sample level notes for bringing in vertical control to a project

(2) VERTICAL CONTROL CHECKS CONT'D — LEVEL LOOP

STA	+	H.I.	−	ELEV.
T.P.#5				7.49 (MLLW)
	5.17	12.66		
T.P.#4			4.24	8.42
	5.41	13.83		
T.P.#3			5.34	8.49
	4.26	12.75		
T.B.M.#1			5.13	7.62
	4.93	12.55		
T.P.#2			4.68	7.87
	4.89	12.76		
T.P.#1			4.87	7.89
	4.55	12.44		
B.M. 4580E 1998			6.54	5.90
	6.69	12.59		
B.M. 4580D 1993			4.17	8.42
	4.14	12.56		
B.M. 4580C 1992			4.11	CALL=8.44 8.45

9-23-04 (3)
P.C. & WARM
JOB #7322
C&C TECHNOLOGIES, INC.
KEN CORMIER - P.C. & NOTES, ⊼
ERIC QUIRK - ROD

RMKS
TOP NUB
TOP NUB - SAME
SAME
SAME
SAME
SAME
SAME
SAME
SAME

NOTE: PLEASE NOTE THAT LEVEL RUN & TIDE STAFF VERTICAL DATUM IS MLLW (MEAN LOWER LOW WATER) EPOCH: 1960-1978
UNITS: FEET US.

236

Figure 3-21. Sample single-wire level notes tying in tidal benchmarks on a dredging project (Jacksonville District)

VERT. DATA TO CHICK TIPS 10

05-27-04

STA.	+	H.I.	−	M.L.L.W ELEV.	STAFF: NGVD.29 ELEV.	~ REMARKS ~
B.M.				7.11	6.46	NGS - Δ - KEY WEST GSL 1989
	6.21	13.32				
TP.#1			5.34	7.98		TOP CONC.
	4.74	12.72				
TP.#2			3.85	8.87		TOP CONC.
	3.74	12.61				
TP#3			3.91	8.70		TOP CONC.
	3.91	12.61				
SET TBM.#1			4.55	8.06		E. END OF 1ST. KLEAT OUT SIDE CONST AREA.
	4.65	12.71				
TP.#4			4.08	8.63		TOP CONC.
	3.84	12.47				
TP#5			4.37	8.10		TOP CONC
	4.54	12.64				
TP.#6			4.65	7.99		TOP CONC.
	5.35	13.34				
B.M.			6.23	7.11		NGS - Δ - KEY WEST GSL 1989
TIED						ERROR: 0.00
TBM#1				8.06		E. END 1ST KLEAT OUTSIDE CONST. AREA.
	4.62	12.68				
TP.#1			4.38	8.30		TOP CONC
	5.13	13.43				

Figure 3-22. Standard single-wire level notes (C&C Technologies, Inc. & Jacksonville District)

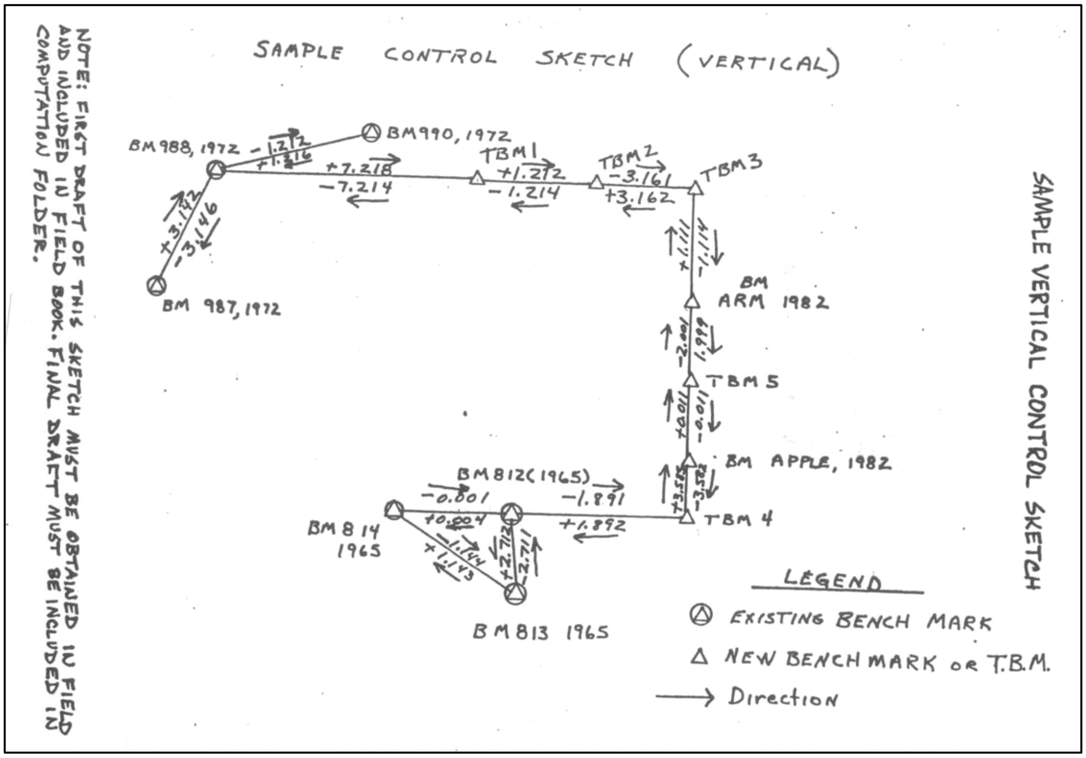


Figure 3-23. Sample vertical control project sketch showing forward and backward level runs for each leg

3-15. Calibrations and Adjustments

To maintain the required accuracy, certain tests and adjustments must be made at prescribed intervals to both the levels and rods being used.

a. Determination of stadia constant. The stadia constant factor of the leveling instrument should be determined by calibration. The stadia factor is required for measurement and computation of distances from the instrument to the leveling rod. This determination is made independently for each level used in the field and is permanently recorded and kept with project files. The determination is made by comparing the measured stadia distance to known distances on a test course.

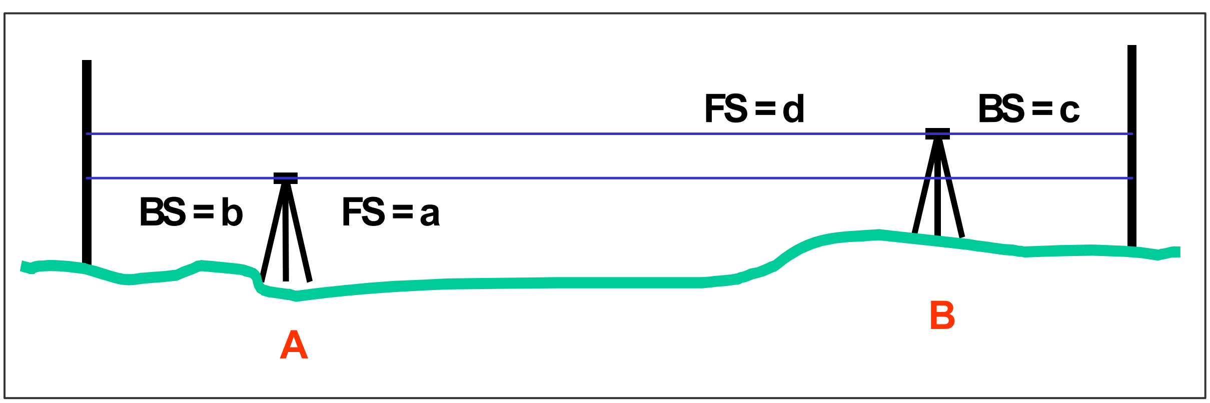


Figure 3-24. C-factor calibration procedure

b. Determination of "C" Factor. Each day, just before the leveling is begun, or just after the beginning of the day's observations, and immediately following any instance in which the level is subjected to unusual shock, the error of the level, or "C" factor, must be determined. This determination can be made during the regular course of leveling or over a special test course; in either case the recording of the observations must be done on a separate page of the recording notes with all computations shown. If the determination is made during the first setup of the regular course of levels, the following procedure is used (Figure 3-24 above). After the regular observations at the instrument station "A" are completed, transcribe the last FS reading "a" as part of the error determination; call up the backsight rodman and have the rod placed about 10 meters from the instrument; read the rod "b" over the instrument to a position "B" about 10 meters behind the front rod; read the front rod "c" and then the back rod "d". The two instrument stations must be between the rod points. The readings must be made with the level bubble carefully centered and then all three wires are read for each rod reading. The required "C" factor determined is the ratio of the required rod reading correction to the corresponding subtended interval, or:

$$C = (R1 - R2) / (R3 - R4) \qquad \text{(Eq 3-8)}$$

where

R1 = Sum near rod readings
R2 = Sum distant rod readings
R3 = Sum distant rod readings
R4 = Sum near rod readings

The total correction for curvature and refraction must be applied to each distant rod reading before using them in the above formula. It must be remembered that the sum of the rod intervals must be multiplied by the stadia constant in order to obtain the actual distance before correction. The maximum permissible "C" factor varies with the stadia constant of the instrument. The instruments must be adjusted if the "C" factor is:

$C > 0.004$ *for a stadia constant of 1/100*
$C > 0.007$ *for a stadia constant of 1/200*
$C > 0.010$ *for a stadia constant of 1/333.*

The determination of the "C" factor should be made under the expected conditions of the survey as to length of sight, character of ground, and elevation of line of sight above the ground. The date and time must be recorded for each "C" factor determination, since this information is needed to compute leveling corrections.

Collimation Check

Project: Example	Location: Fort Belvoir, Virginia	Organization: DMS			
Observer: SFC Jones	Recorder: SGT Smith	Instrument: WILD NA2-1234	Sun: Clear	Wind: Calm	Weather: Warm
From	To	Date: 2000 09 30	Time: 0813-0835	Line or Net: Belvoir Net 1	Page No. / No. of Pgs.

Station	Backsight Face of rod	Mean	Back of rod	Interval	Sum of Intervals	Foresight Face of rod	Mean	Back of rod	Interval	Sum of Intervals	Remarks
	1821					1055					STADIA
	1771	1771.0		050		0680	0679.7		375		CONSTANT
	1721			050	100	0304			376	751	0.100
	5313	1771.0			100	2039	0679.7			751	
	1476					2908					
	1432	1432.0		044		2530	2530.7		378		
	1388			044	088	2154			376	754	
	9609	3203.0			188	9631	3210.4			1505	
						C & R #1	-0.4		-	188	
MEAN MIDDLE WIRE						C & R #2	-0.4		SUM (1)	1317	
SUM			SUM MEAN MIDDLE WIRE (FORESIGHT)				-3209.6				
ROD CORRECTIONS			SUM MEAN MIDDLE WIRE (BACKSIGHT)				3203.0			±	
CURVATURE & REFRACTION					SUM (1)	1317	-6.6		C =	-.0050	
NOTE:										*754	
CORRECTION IS IN ROD										-3.8	CORR TO MIDDLE WIRE
UNITS										+2530.0	
										2526.2	CORR ROD READING
SIGHT DISTANCE										2.526	

METERS	YARDS	C & R
00	00	
27.0	28.2	00
46.8	48.9	0.1
60.4	63.1	0.2
71.4	74.7	0.3
81.0	84.7	0.4
89.5	93.6	0.5
97.3	101.8	0.6
104.5	109.3	0.7
111.3	116.4	0.8

FOR CLASSROOM PURPOSES ONLY

INST OP INT

1st COMP INT

2nd COMP INT

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Figure 3-25. Example of C-factor computation (FM 3-34.331)

c. Adjustment of level. The type of instrument being used will dictate the method and procedure used to adjust the instrument if the "C" factor exceeds the allowable limits. The manufacturer's procedures should be followed when adjusting a level.

d. Test of rod levels. Precise rod levels must be tested once each week during regular use--or whenever they receive a severe shock. This test is made with the level rod bubble held at its center, and the deviation of the face and edge of the rod from the vertical are determined. If the deviation from the vertical exceeds 0.01 meter on a 3-meter length of rod, the rod level must be adjusted. The rod level is adjusted in the same manner as any other circular bubble. A statement must be inserted in the records showing the manner in which the test was made, the error that was found, if any, and whether an adjustment was made. When using other than precise leveling rods, this test is not required.

3-16. Care of Level Instruments

The following sections on care and maintenance of leveling equipment are taken from the CALTRANS *Surveys Manual* (CALTRANS 2002-2004). This guidance is applicable to Corps field operations.

a. Optical Pendulum Level. Pendulum levels are fast, accurate, and easy to maintain. Proper care and service is required to provide continuous service and to maintain precision in measurement. Never disassemble an instrument in the field. Only make those adjustments outlined in the operator's manual.

Care of a Pendulum Level:

- To prevent compensator damage, do not spin, bounce, or hit pendulum levels.
- Protect the level from dust. Dust or foreign matter inside the scope can cause the compensator damping device to hang up.

b. Circular bubble test and adjustment. Frequently check adjustment of the bullseye bubble. Adjust the bubble to the center of the bullseye. Make certain the bubble is adjusted along the line of sight and 90° to the line of sight as well. Proper adjustment reduces the possibility of compensator hang-up. Adjustment will be easier if done in the shade, where temperature is constant.

c. Horizontal cross-hair test and adjustment (Two-Peg Test). At least once every 90 days or when discrepancies show up in the leveling work and before every three-wire level survey, the "Two-Peg Test" should be performed as follows:

- Select two benchmarks "A" and "B" approximately 60 m apart.
- Set up the level midway between the two points "A" and "B" and record the rod readings of each point determining their difference in elevation.
- Move the level 6 m beyond either benchmark and record the rod reading of both points again, once again determining their difference in elevation.
- If the difference in elevation measured at each setup is the same, the level is in adjustment. If not, the horizontal cross-hair should be adjusted as detailed in the operator's manual.
- After the adjustment repeat the "peg test" again to check the adjustment.

d. Mechanical functions. To check for compensator hang-up, lightly tap the telescope with a pencil or operate the fine movement screw jerkily to and fro. If the compensator is slow to respond or malfunctioning, send the instrument to an approved repair service. There are no mechanical field adjustments that can be made on the compensator.

e. Electronic Digital Bar-Code System. Digital bar-code levels (Figure 3-26) operate by comparing the observed digital image of a bar-code leveling rod with a map of the bar code stored in the level's memory. These instruments are also equipped with a conventional pendulum-type compensator and may be used as an optical level. An on-board computer processes all leveling operations including determination of sight lengths. A bar-code system should include:

- Digital level with data recorder module or cable connected data collector
- Data reader and/or appropriate computer interface
- Bar-code leveling rods

(1) Field operations. At the beginning and end of each day's operation, check the instrument for collimation error, recording the tests into the survey notes. When using electronic digital leveling instruments, the absolute collimation error will be recorded along with the leveling data. If an error in excess of 2 mm within a 60 m sight distance is detected, the level should be readjusted. If the instrument is severely jolted or bumped, or suspected as such, it should be immediately checked. Manufacturers' specifications state that the electronic digital leveling instrument should not be exposed to direct sunlight and recommend use of an umbrella in bright sunlight.

(2) Horizontal cross-hair test and adjustment (Two-Peg Test). The test and adjustment procedure for an electronic digital level is geometrically similar to the two peg procedure for a conventional optical

level. However, all horizontal and vertical measurements and differences are measured and recorded electronically. The collimation error is recorded by the on-board computer.

(3) Data collection, storage, and transfer. Raw data generated by an electronic digital level is stored in a data collector and processed into field book format. Software will perform simple or least-squares adjustment of the data. An ASCII file may be created that can be imported into road design software.

(4) Leveling rods. Leveling rods should be maintained and checked as any other precision equipment. Accurate leveling depends as much on the condition of the rods as on the condition of the levels. Reserve an older rod for rough work, such as measuring inverts, mud levels, water depths, etc.


Figure 3-26. Digital Bar Code Level--Wild NA2 (New Orleans District)

3-17. Routine Maintenance and Care of Level Rods

a. Maintenance procedures common to all types of rods are.

- Periodically lubricate hardware and slip joints with an oil-free silicon spray.
- Clean sand and grit from slip joints.
- Clean graduated faces with a damp cloth and wipe dry.
- Keep the base plate clean.
- Periodically check all screws and hardware for snugness and operation.
- Periodically check accuracy by extending rod and measuring between graduations across rod section divisions with an accurate tape.

b. Transport and storage.

- If possible, leave a wet rod uncovered and extended until it is thoroughly dry.
- Store rods in protective sleeves or cases, in a dry location, either vertically (not leaning), or horizontally. When stored horizontally, either fully support the rod or provide at least three-point support.

c. Field operations.

- Touch graduated faces only when necessary and avoid laying the rod where the graduated face will come into contact with other tools, objects, or materials that could mar the face.
- Do not abuse a rod by throwing, dropping, dragging, or placing it where it might fall.
- Do not lay a rod in sand, dust, or loose granular material.
- Lower rod sections as the rod is being collapsed. Do not let them fall or drop.

d. Direct reading rod. At frequent intervals, check all components for wear. Periodically lubricate all hardware, racks, and rollers with an oil-free silicone aerosol spray. If the tape guides begin to snag or bind the tape, have the rod repaired.

e. Fiberglass leveling rod. Dowels through the bottom of each section keep the section above from falling inside the lower section. Dropping the sections when collapsing the rod will loosen the dowels causing the sections to jam and may also shatter the fiberglass around the dowel holes. Observe the following precautions:

- When the slip joint goes bad, remove the rod from service.
- Lubricate fiberglass rods with an oil-less silicone spray or with talcum.

f. Invar leveling rods. Invar rods are precisely made and standardized; extra care is required to maintain this precision. Observe the following precautions:

- Store, fully supported and stopped, in a water-proof case.
- Do not use invar rods in rain or dust.
- Carry parallel with the ground, in alternate “face-up” and “face-down” positions to equalize weight stresses.
- Avoid laying an invar rod on the ground.
- If foreign matter has “fouled” a rod, carefully disassemble and clean.
- The rod tape must slide freely in the recessed guides as the wooden staff swells or shrinks.

g. Bar-code leveling staffs (Rods). A typical bar-code leveling staff is of a different design and construction than a conventional level rod. Several types of bar-code rods are available, depending on the type of work performed. Designs range from an aluminum/invar-tape, precise staff to various sectional staffs constructed of wood, aluminum, or fiberglass. Care and maintenance of these staffs is minimal due to their simplistic construction. Store in clean, dry condition and always transport in carrying cases.

3-18. Maintenance of Survey Instrument Accessories

a. Tripods. Tripods support and provide a fixed base for all types of surveying instruments. The typical tripod has a 5/8-in. x 11 thread fastener to secure an instrument or accessory to the tripod head. The head provides a lateral adjustment range for the instrument of approximately 25 mm. The tripods are of a wide-frame design and have extendible legs. A secure and stable tripod is required for the support of precision instruments. There should be no slack between the various components of a tripod. Loose joints or fittings will cause instability. Some guidelines to properly maintain tripods are:

- Maintain a firm snugness in all metal fittings. Over-tightening is the cause of crushed wooden components and stripped threads.
- Tighten leg hinges just enough to support the fully extended legs when a tripod is lifted clear of the ground.

- Keep the metal tripod shoes tight and free of dirt and debris.
- Keep wooden parts of tripods well painted or varnished to reduce swelling and shrinking due to moisture content of the wood.
- Always replace top caps when tripod is not in use to protect the mounting surface and head from damage.
- Use care when placing or removing tripods from the survey vehicle, as significant damage can occur. Ensure that carry compartments are designed and constructed to isolate tripods from each other and from other equipment.

b. Tribrachs. Tribrachs are the detachable base for most survey instruments and many accessories. They are equipped with an optical plummet and spherical "bullseye" level. The ability to "leapfrog" instrument setups by interchanging instruments, prisms, targets, or antennas without disturbing the setup of a tribrach greatly enhances the speed, efficiency, and accuracy of a survey. Some guidelines to properly maintain tribrachs are:

- Transport tribrachs in separate compartments or containers to prevent damage to the base surfaces, spherical level, and optical plummet.
- Do not over-tighten the tripod fastener screw.
- Clean leveling screws regularly.
- When tribrachs are not in use, set leveling screws at mid-range, usually marked by a horizontal line.
- Use care whenever using range poles mounted on a tribrach to vertically extend a sight, antenna, or prism. Extensions place considerable stress on the leveling plate.
- Adjust spherical level and vertical collimation of optical plummet routinely.

c. Prism Poles/Antenna Poles. An attached adjustable spherical level bubble (bullseye) is used to maintain a prism/antenna pole in a vertical position. A maladjusted level bubble may cause systematic error when using the pole. A simple method for checking the accuracy of the bullseye bubble is to check the rod by placing it against a door jamb or other permanent vertical part of a building that has been previously verified as being vertical.