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Course #:	SUR-110
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SUR-110 EXAM PREVIEW

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Exam Preview:

1. "Topographic surveying" encompasses a ___ range of surveying and mapping products, ranging from aerial mapping to ground and underground surveys.
   a. Specialized
   b. Direct
   c. Narrow
   d. Broad
2. The type of survey, map scale, and contour interval should be selected in each case to interpret the character of the terrain most suitably for the purpose, and the ____ of permissible error should be prescribed in each instance.
   a. Accuracy
   b. Source
   c. Control
   d. Tolerance
3. Determination of plan scales - the scale of topographic maps should be chosen so that these maps may serve as base maps over which subsequent project drawings can be drawn at the same scale.
   a. True
   b. False
4. Purpose of detailed topographic surveys. Detailed topographic surveys are the basis for detailed plans showing the site layout and utilities. The area to be covered by detailed surveys should be maximized to serve the requirements of the actual building area and should not be made where reconnaissance surveys will serve.
   a. True
   b. False

5. There are several kinds of utility surveys, but principally they can be divided into two major types. One type is performed for the layout of new systems, and the other is the location of existing systems.
   a. True
   b. False
6. ____ surveys are surveys compiled to show actual condition of completed projects as they exist for record purposes and/or payment. Since many field changes occur during construction, both authorized and clandestine, surveys are regularly completed to check the project against plans and specifications.
   a. Horizontal
   b. Vertical
   c. Plat
   d. As-built
7. Prior to the advent of total stations, GPS, LIDAR, and data collector systems, transit and plane table topographic surveying methods and instruments were once standard.
   a. True
   b. False
8. Total stations were first developed in the 19__s by Hewlett-Packard (Brinker and Minnick 1995). These instruments sensed horizontal and vertical angles electronically instead of optically, and combined them with an EDM slope distance to output the X-Y-Z coordinates of a point relative to the instrument's X-Y-Z coordinates.
   a. 50
   b. 60
   c. 70
   d. 80
9. Periodically lubricate hardware and slip joints with an oil-free silicon spray is a maintenance procedure common to all types of Level Rods.
   a. True
   b. False
10. ____is an arbitrarily chosen angle between the plane of the celestial meridian and the plane of an initial meridian. Astronomical longitude is the longitude that results directly from observations on celestial bodies, uncorrected for deflection of the vertical.
   a. Astronomical Longitude
   b. Astronomical Latitude
   c. Astronomical Triangle
   d. Angular Misclosure

Chapter 1
Introduction

1-1. Purpose

This manual provides guidance on performing detailed site surveys of military installation facilities and civil works projects. Technical specifications, procedural guidance, and quality control criteria are outlined for developing large-scale site plans used for engineering drawings of planned projects, or detailed as-built feature mapping of completed facilities.

1-2. Applicability

This manual applies to all USACE commands having responsibility for the planning, engineering and design, operation, maintenance, construction, and related real estate and regulatory functions of military construction, civil works, and environmental restoration projects. It is intended for use by hired-labor personnel, construction contractors, and Architect-Engineer (A-E) contractors. It is also applicable to surveys performed or procured by local interest groups under various cooperative or cost-sharing agreements.

1-3. Distribution

This publication is approved for public release; distribution is unlimited.

1-4. References

Referenced USACE publications and related bibliographic information are listed in Appendix A. Where applicable, primary source material for individual chapters may also be noted within that chapter.

1-5. Discussion

Control and topographic surveys are performed to determine the planimetric location and/or elevation of surface or subsurface features, facilities, or utilities. These surveys are normally used to prepare highly detailed site plan maps (and digital databases) of a project site, facilities, or utility infrastructure; for future design, on going construction, or as-built condition. Engineering drawing scales are typically large--ranging between 1 inch = 30 ft and 1 inch = 100 ft. These surveys are performed over relatively small project sites using tripod-mounted, manually operated, terrestrial survey equipment, such as transits, tapes, levels, plane tables, electronic total stations, and GPS receivers. This manual covers the field survey techniques, instrumentation, and electronic data collection systems that are used in performing these ground-based field surveys, and transferring observed data into facility management or design databases. Also included are methods for extending geodetic control needed for supplemental topographic mapping work on a military installation or civil works project site. This manual also includes procedures for transferring field data to computer-aided drafting and design (CADD) systems or geographic information systems (GIS) used in planning, engineering, construction, and facility management. Aerial topographic mapping techniques are not addressed in this manual--see EM 1110-1-1003 (*Photogrammetric Mapping*). FM 3-34.331 (*Topographic Surveying*) should be consulted for tactical field surveying operations supporting field artillery (FA), air-defense artillery (ADA), aviation (e.g., airfield NAVAID and obstruction surveys), intelligence, communications, or construction.

1-6. Use of Manual

This manual is intended to be a USACE reference guide for control surveying, site plan mapping, and infrastructure utility feature mapping. These activities may be performed by hired-labor forces, contracted forces, or combinations thereof. This manual will also used as the primary reference manual for Proponent Sponsored Engineer Corps Training (PROSPECT) courses on topographic surveying. Accuracy specifications, procedural criteria, and quality control requirements contained in this manual may be directly referenced in the scopes of work for Architect-Engineer (A-E) survey services or other third-party survey services, including construction contracts. This is intended to assure that uniform and standardized procedures are followed by both hired-labor and contract service sources throughout USACE.

Figure 1-1. PROSPECT topographic survey training course “Surveying III”
USACE Professional Development Support Center, Huntsville, AL (2003)

1-7. Scope of Manual

The overall scope of this manual is limited to ground-based survey methods--specifically, georeferenced observations taken from survey instruments set up on tripods over fixed control points or benchmarks. These methods usually provide the highest accuracy for engineering surveys, and are necessary when surface and subsurface utilities must be definitively located and identified. Therefore, less-accurate and less-detailed remote aerial mapping techniques are excluded. However, ground-based topographic surveys covered in this manual are normally required to supplement generalized aerial topography, and to provide feature details on surface and subsurface infrastructure. Control survey applications are limited to establishing supplemental (or secondary) reference points at a project site from which detailed topographic mapping or construction stake out is performed. Geodetic control survey methods used for high-order densification of the national reference network (i.e., the National Spatial Reference System--NSRS) are not covered in this manual--see EM 1110-1-1003 (*NAVSTAR GPS Surveying*) for performing precise geodetic control surveys.

a. Technical references. Technical or procedural guidance is in more general terms where methodologies are described in readily available references or in survey instrumentation and data collector operation manuals. This manual does not duplicate elementary surveying topics that are adequately covered in a number of academic texts, such as those recommended in paragraph A-2 at Appendix A. References to these publications will be provided to avoid unnecessary redundant coverage of elementary topics. It is strongly recommended that a user performing control or topographic surveys acquire one of these textbooks. Topics that will be referenced include instrument set-up and operation, taping and chaining, instrument calibrations, basic surveying theory and accuracy estimates, traditional survey distance, curve and area computations, traverse adjustments, etc. Reference is also made to various survey and CADD manuals published by State Departments of Transportation (DOT). An excellent example is the California Department of Transportation *Surveys Manual* (CALTRANS Surveys Manual 2001-2004), a 14 chapter publication that can be downloaded at the CALTRANS web site www.dot.ca.gov/hq/esc/geometronics. Chapters in the CALTRANS *Surveys Manual* are periodically updated as new equipment or techniques are developed. Since design and construction surveys performed by many State DOTs are nearly identical to Corps of Engineers applications, these DOT manuals represent an excellent up-to-date resource on procedures, instrumentation, standards, and specifications.

b. Manual coverage and appendices. The first few chapters in this manual are intended to provide a general overview of control and topographic survey procedures, equipment, and standards. Reference systems and datums used on Corps civil projects and military installations are described in Chapter 5. Subsequent chapters cover survey planning, data collection, data processing, and generation of digital or hard copy site plans. Examples of different topographic survey methods are included--e.g., total station, LIDAR, GPS. Sample topographic survey projects are included within chapters or appendices. Examples include those surveys typically performed on Corps civil works or military construction projects, such as navigation, flood control, real estate, facility design and maintenance, lock and dam surveys, and utility surveys. The final chapter on estimating costs for topographic surveys is intended to assist those USACE commands that contract out these services. Sample scopes of works to contracted and hired-labor field personnel are provided, including data deliverable requirements, are provided in this chapter and in the appendices. The appendices also include a number of Corps project applications where topographic surveys were performed in support of civil and military design and construction.

c. Evolving technology and procedures. Survey equipment operation, calibration, and procedural methods for acquiring, logging, processing, and plotting topographic survey data are adequately detailed in operation manuals provided by the various instrument manufacturers and software vendors. Since instrument and data collector operations (and data processing methods) are unique to each vendor, and are being constantly updated, this manual can only provide a general overview of some of the more common techniques used by the Corps or its contractors. As new survey instruments, technology, and machine control integration procedures are developed, Districts are strongly encouraged to use those innovations and recommend modifications to any criteria or technical guidance contained in this manual--see Proponency and Waivers section at the end of this chapter. Other Corps regulations may dictate mandatory requirements for processing, displaying, transferring, and archiving survey data--e.g., metadata archiving. These regulations will be referenced where applicable.

d. Manual development. Technical development and compilation of this manual was coordinated in 2004-2005 by the US Army Engineer Research and Development Center--Topographic Engineering Center (CEERD-TR-A). The following USACE Districts provided project examples and/or performed technical reviews on various drafts of the manual: Jacksonville, Louisville, New Orleans, Pittsburgh, Philadelphia, Portland, Rock Island, St. Louis, Tulsa, and Walla Walla. The original version of this manual (*Topographic Surveying*) was developed in the early 1990s by the USACE Topographic Engineering Center at Fort Belvoir, and published on 31 August 1994. This latest update consolidates control surveying topics from EM 1110-1-1004 (*Geodetic and Control Surveying*), dated 1 Jun 02. EM 1110-1-1004 is superseded by this consolidation into EM 1110-1-1005.

1-8. Life Cycle Project Management Applicability

Project control established during the planning phase of a project may be used through the entire life cycle of the project, spanning decades in many cases. During initial reconnaissance surveys of a project, primary control should be permanently monumented and situated in areas that are conducive to the performance or densification of subsequent surveys for contract site plans, construction, and maintenance. During the early planning phases of a project, a comprehensive survey control plan should be developed which considers survey requirements over a project's life cycle, with a goal of eliminating duplicative or redundant surveys to the maximum extent possible.

1-9. Metrics and Accuracy Definitions

Both English and metric units are used in this manual. Metric units are commonly used in survey instrumentation, such as electronic distance measurement and in GPS surveys. Metric-derived geographical or metric Cartesian coordinates are transformed to English units of measurements for use in local project reference and design systems, such as State Plane Coordinate System (SPCS) grids. In all cases, the use of metric units shall follow local engineering and construction practices. English/metric equivalencies are noted where applicable, including the critical--and often statutory--distinction between the US Survey Foot (1,200/3,937 meters (m) exactly) and International Foot (30.48/100 m exactly) conversions. One-dimensional (1D), two-dimensional (2D), and three-dimensional (3D) accuracy statistics, standards, and tolerances specified in this manual are defined at the 95% RMS confidence level. Unless otherwise stated, "positional accuracies" imply horizontal (2D) RMS measures.

1-10. Trade Name Exclusions

The citation or illustration in this manual of trade names of commercially available products, including supporting surveying equipment, instrumentation, and software, does not constitute official endorsement or approval of the use of such products.

1-11. Abbreviations and Terms

Abbreviations, acronyms, and engineering surveying terms used in this manual are explained in the Glossary at the end of this manual.

1-12. Mandatory Requirements

ER 1110-2-1150 (*Engineering and Design for Civil Works Projects*) prescribes that mandatory requirements be identified in engineer manuals. Mandatory accuracy standards, quality control, and quality assurance criteria are normally contained in tables within each chapter, and these requirements are summarized at the end of the chapter. If no mandatory requirements are listed, then the material in a particular chapter is considered recommended guidance. Any mandatory criteria contained in this manual are based on the following considerations: (1) project safety assurance, (2) overall project function, (3) previous Corps experience and practice, (4) Corps-wide geospatial data standardization requirements, (5) adverse economic impacts if criteria are not followed, and (6) HQUSACE commitments to industry standards.

1-13. Governing Engineer Regulations and Related Standards

Spatial coordinates established using topographic survey techniques fall under the definition of geospatial data contained in ER 1110-1-8156 (*Policies, Guidance, and Requirements for Geospatial Data and Systems*). Accordingly, the guidance in ER 1110-1-8156, and its implementing manual EM 1110-1-2909 (*Geospatial Data and Systems*), must be followed for disseminating and archiving survey data. This would include preparing appropriate metadata files in accordance with the guidance in EM 1110-1-2909. Detailed CADD and GIS standards are promulgated by the CADD/GIS Technology Center in Vicksburg, MS. Federal standards for reporting survey accuracy, geodetic control survey standards, and topographic survey standards are also published by the Federal Geographic Data Committee (FGDC). These FGDC "*Geospatial Positioning Accuracy Standards*" are listed in Appendix A.

1-14. Proponency and Waivers

The HQUSACE proponent for this manual is the Engineering and Construction Division, Directorate of Civil Works. Comments, recommended changes, or waivers to this manual should be forwarded through MSC to HQUSACE (ATTN: CECW-CE).

Chapter 2
Overview of Topographic Surveying Techniques and Methods

2-1. Purpose

This chapter provides an overview of the types of surveys and equipment that are used for performing control and topographic surveys. It covers the various types of engineering surveys used to support facility design and construction, the different survey equipment and instruments, and the general field and office procedures that are performed.

Figure 2-1. Topographic survey at Prompton Dam, PA (Philadelphia District--April 2005)

2-2. General Definitions

"Topographic surveying" encompasses a broad range of surveying and mapping products, ranging from aerial mapping to ground and underground surveys. "Control surveying" likewise can cover wide area geodetic surveys to construction stakeout. The following definitions from the Florida Administrative Code (FAC 2003) illustrates how topographic and control surveying falls under the overall "surveying and mapping" field:

> "Surveying and Mapping: a process of direct measurement and analysis specifically designed to document the existence, the identity, the location, and the dimension or size of natural or artificial features on land or in the air, space or water for the purpose of producing accurate and reliable maps, suitable for visualization if needed, of such documentation."

> "Topographic Survey: a survey of selected natural and artificial features of a part of theearth's surface to determine horizontal and vertical spatial relations."

> "Control Survey: a survey which provides horizontal or vertical position data for the support or control of subordinate surveys or for mapping."

Anderson and Mikhail 1998 define topographic surveys as follows:

> "A topographic map shows, through the use of suitable symbols, (1) the spatial characteristics of the earth's surface, with such features as hills and valleys, vegetation and rivers, and (2) constructed features such as buildings, roads, canals, and cultivation. The distinguishing characteristic of a topographic map, as compared with other maps, is the representation of the terrain relief."

As outlined in the scope in Chapter 1, the guidance and instructions in this manual will focus on the performance of site plan surveys required for the design and construction of military facilities and civil works projects. Control survey methods will focus on those surveys required to support detailed site mapping. ERDC/ITL 1999b defines installation map products as:

> "Maps are tools that provide the commanders with timely, complete, and accurate information about our installation. They have three primary uses on the installation: locate places and features, show patterns of distribution (natural and physical phenomena), and compare and contrast map information by visualizing the relationships between these phenomena. A map is a representation of reality comprised of selected features needed to meet the maps intended purpose."

2-3. Generic Considerations Applicable to all Drawings and Maps

There are some differences between topographic maps and plans (or engineering drawings). In general, maps are usually developed at a smaller scale, whereas detailed site plan drawings are at much larger scales. The following table (from Kavanagh 1997) illustrates this distinction:

Table 2-1. Map Scales Used for Various Engineering Drawings

Scale	Typical Uses
1" = 1' to 1" = 8'	Large scale detail drawings, architectural plans
1" = 20' -30'- 50' to 1" = 100'	Engineering site plans, facility design
1" = 100' to 1" = 800'	Intermediate scale: planning studies, drainage, route planning
1" = 1,000' and smaller	Small scale: topographic maps, USGS quad maps, NOAA charts

In general, map scales greater than 1 inch = 100 ft are intended for detailed design purposes. Smaller scales between than 1 inch = 100 ft and 1 inch = 1,000 ft are used for general planning purposes. To assure convenience of use and to derive full benefit from maps and plans, the data on various types of the above project drawings should contain the following criteria:

a. State Plane Coordinate System. It is desirable that the surveys and construction drawings for different projects be correlated with each other and with other Federal agencies. This is accomplished by the use of the State Plane Coordinate System as the coordinate system of the project, and insuring that this grid reference is adequately tied in to a nationwide geodetic reference system. In some cases, an arbitrary or artificial coordinate system may be used.

b. Coordinate grid lines. To enable proper correlation between the various project maps and plans, there must be drawn on all project maps (or CADD sheet files) the coordinate grid lines of the project coordinate system. These lines should be spaced five (5) inches apart; the outside coordinate lines should be the match lines for adjacent map sheets.

c. Determination of plan scales. The scale of topographic maps should be chosen so that these maps may serve as base maps over which subsequent project drawings can be drawn at the same scale. Reference Table 6-1 for guidance on selecting map scales. Thus, the scale of a reconnaissance topographic survey is chosen at a convenient scale so that it may serve as base over which the preliminary site studies and approved site plan may be prepared. The detailed topographic maps will also serve as a base upon which to prepare detailed utility maps.

d. Outside sources of information. Full use should be made of surveying and mapping information available in the various Federal, state, local agencies, and geospatial data clearinghouses prescribed in Corps regulations. USACE Commands are required by ER 1110-1-8156 to perform clearinghouse searches.

e. Accuracy. The type of survey, map scale, and contour interval should be selected in each case to interpret the character of the terrain most suitably for the purpose, and the tolerance of permissible error should be prescribed in each instance. It is not necessary that reconnaissance topographic surveys be of the same degree of accuracy as detailed topographic surveys, nor should they show the topographic data with the same degree of detail.

f. Control to be shown on Plans. The coordinate grid, horizontal control stations, benchmarks, and related reference datums should be shown on all maps and plans--see Chapter 5 for specific details. This is of particular importance in detailed layout plans of building areas drawn at the larger scales of 1 inch = 100 ft or 1 inch = 50 ft. The data are vital to speed and accuracy in subsequent location survey work.

SECTION I
Types of Surveys

The following types of products are covered by this manual, all of which are assumed to fall under the broad definition of "topographic surveying":

- Reconnaissance Topographic Surveys
- Detailed Topographic Surveys and Maps
- Utility Surveys and Maps
- As-built Drawings
- Boundary Surveys and Reservation Maps
- Reservoir Clearing Surveys
- Upland Disposal Area Surveys
- Channel Improvement and Cutoff Surveys
- Post-Flood High Water Mark Surveys
- Bridge Surveys
- Artillery Surveys
- Airport Obstruction and NAVAID Surveys
- Site Plans (Hydrographic, Beach, Levee, Route, Quantity, Structure, etc.)
- Army Installation Drawings

Some of the above surveying and mapping applications will overlap in practice and definitions will vary from District to District. Aerial topographic mapping products are excluded from this manual, as are high-order geodetic control surveys.

2-4. Reconnaissance Topographic Surveys

Topographic surveys have various definitions by different agencies and publications. These may include everything from photogrammetric mapping to hydrographic surveys. The reconnaissance topographic surveys described below relate to smaller scale preliminary mapping performed in advance of engineering and design, and are often called preliminary surveys. Following are the important considerations in connection with reconnaissance topographic surveys:

a. Purpose of reconnaissance surveys. The reconnaissance survey is the basis for a general study or a decision as to the construction suitability of areas. It may also be used for preliminary site layouts. Reconnaissance surveys are useful in showing the general location of roads, building areas, and utilities; and to establish an acceptable site layout which must be approved by authorized officers before detailed layout plans can be made. Such surveys also enable the proper selection of those areas, relatively limited in extent, which should be covered by the more time-consuming and costly detailed topographic surveys. In some instances the US Geological Survey (USGS) topographic quadrangle sheets may be enlarged and used for this purpose. The success of such use will be dependent upon the contour interval, whether the USGS maps are of recent date, the character of the terrain, and the nature of the project.

b. Map scales and contour intervals. Dependent upon the size and shape of the area and upon the nature of the terrain, i.e., density of culture and steepness of slope, reconnaissance surveys may be at scales varying from 1 inch = 400 ft to 1 inch = 1,000 ft. In cases where the project is of limited size, a scale of 1 inch = 200 ft may be used. Contour intervals of either five feet or ten feet may be used. The five-foot interval is the more serviceable and should be used except where steepness of slope makes the ten-foot interval advisable. When areas contain both flat and very steep slopes a ten-foot interval may be adopted as the contour interval of the map. On the flat areas, one-half interval contour (e.g., five foot) may be shown, discontinuing them wherever the slopes become steep or uniform. Contours having different intervals should not be shown by the same symbol on the same map. In extremely flat areas, a one- or two-foot contour interval may be required to adequately represent the terrain.

c. Accuracy and degree of detail. Extreme accuracy of position is not necessary and minutiae of detail are not desirable. The map should show all pertinent physical features such as roads, railroads, streams, cleared and wooded areas, houses, bridges, cemeteries, orchards, lakes, ponds, and fence lines. Elevations should be shown by contours and spot elevations at road intersections, bridges, water surfaces, tops of summits and bottoms of depressions.

d. Datum. When practical and feasible, it is desirable for these surveys to be referenced to an established NSRS datum, rather than some arbitrary grid system.

2-5. Detailed Topographic Surveys

Following are the important considerations in connection with detailed topographic surveys. Further guidance is contained in Chapter 6.

a. Purpose of detailed topographic surveys. Detailed topographic surveys are the basis for detailed plans showing the site layout and utilities. The area to be covered by detailed surveys should be kept to a minimum to serve the requirements of the actual building area and should not be made where reconnaissance surveys will serve. Detailed topographic surveys may be made by plane table, total station, GPS, laser scanning, and/or photogrammetry.

b. Map scales and contour intervals. Detailed topographic surveys should be at a map scale of 1 inch = 100 ft or 1 inch = 200 ft, with contour intervals of two feet, depending on the convenient size to be established for detailed site plans and utility maps. A scale of 1 inch = 50 ft is also used on small projects.

c. Accuracy and degree of detail. Upon the map sheets there shall be shown all control points and bench marks with their designating numbers and their elevations, all roads, railroads, streams, fence lines, utilities, poles, isolated trees ten inches or more in diameter, boundaries of timbered areas rock ledges or boulders, wells, buildings, cemeteries and any other physical data that will affect planning. In addition to elevations shown by the contours, there should be shown spot elevations at all summits, bottoms of depressions, tops of banks, stream or water surfaces, roads and railroad lines at breaks of grade, intersections, bridges, bases of isolated trees, first-floor elevation of existing buildings, and ground surfaces at wells. The contours should faithfully express the relief detail and topographic shapes. Accuracy standards for topographic mapping features are detailed in Chapter 4.

d. Horizontal control. There should be established a system of monumented horizontal control originating from and closing upon existing NSRS control points. Since this control should also serve the needs of subsequent site layouts and utility maps, the selection of its position and frequency must give due weight to these needs. In areas where there is to be intensive development, the lines of control circuits should ideally not be more than 2,000 to 2,500 feet apart in one direction, but may be of any convenient dimension in the other direction. Control points should generally not be more than 800 to 1000 feet apart along the line of the circuit and should be intervisible. In order to serve property survey needs, the outside control circuits should have control points within 300 or 400 feet of probable property boundary corners. Where topography is to be taken by plane table, a sheet layout should first be made and the control circuits selected near two sheet borders so that the line may be platted on both sheets.

e. Vertical control. Vertical control should consist of levels run in circuits originating from and closing upon Federal Government benchmarks. The closure error of these circuits should be predicated on the character and scope of construction involved. The elevation of each traverse station monument should be determined. Other permanent benchmarks as deemed necessary should be set.

f. Reference datum. When practical and feasible, it is desirable for these surveys to be referenced to an established NSRS datum, rather than some arbitrary grid or vertical reference system.

2-6. Utility Surveys

There are several kinds of utility surveys, but principally they can be divided into two major types. One type is performed for the layout of new systems, and the other is the location of existing systems. Typical utilities that are located include communications lines, electrical lines, and buried pipe systems including gas, sewers, and water lines. The layout of new systems can be described as a specialized type of route surveying, in that they have alignment and profiles and rights-of-ways similar to roads, railroads, canals, etc. In reality, utilities are transportation systems in their own right. Utilities are special in that they may have problems regarding right-of-way above or below ground. A great portion of utility surveying involves the location of existing utilities for construction planning, facility alteration, road relocations, and other similar projects. This is a very important part of the preliminary surveys necessary for most of these projects.

a. Purpose of utility maps. To a greater degree than any other drawings prepared in the field, utility maps serve two main purposes: (1) as construction drawings, and (2) as permanent record of the utilities, i.e. "as built." Their value to the Facility or Public Works Engineer in the proper operation and maintenance of the project is such as to require complete information on all pertinent features of each utility. For the purpose of recording "as built" construction in the most readily usable form, two sets of utility maps are usually found most practicable: General Utility Maps at small scale (1 inch = 400 ft or 1 inch = 200 ft and Detailed Utility Maps (sometimes referred to as unit layout maps) at a larger scale (1 inch = 100 ft or 1 inch = 50 ft) as described below.

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

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

(2) Proper identification of utilities sometimes takes an expert, particularly regarding buried pipes. There are many types of wire lines on poles and in below ground conduits--this can lead to identification problems. Where once only power and telephone lines were of concern, now cable TV, burglar alarms, and even other wire or fiber optic line types must be considered.

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

c. General utility maps. At a scale of 1 inch = 400 ft, 1 inch = 300 ft, or on smaller projects at scales of 1 inch = 200 ft, the principal features of each utility are generally shown separately, each utility on a separate map, or CADD layer/level. As a base upon which to add the data for each utility it is normally most convenient to use the approved general site plan of the same scale. The amount of detail to be indicated on each general utility map should be limited to that consistent with the scale of the map. It is usually feasible, even at a scale of 1 inch = 400 ft, to indicate the location, material, pipe sizes, etc., of the main distribution and collection systems, leaving minor construction features (valves, service connections, etc.) to be shown at larger scale on the second set of maps (see Detailed Utility Maps below) or as attributes in a CADD facilities database. General Utility Maps will normally include the following:

- Water Distribution System.
- Sewers: Sanitary and Storm Water Collection Systems
- Electric Distribution System, including Fire Alarm System.
- Communication Systems (telephone, cable, computer, fiber optic, etc.)
- Gas System.
- Gasoline Storage and Fuel System.
- Steam distribution and domestic hot water system for hospitals and other areas. (Include compressed air system, if any)
- Target and Magazine Areas. (Where located at some distance from the general construction area, these may be shown on a separate map annotated to show the relation to the project)
- Rail Facilities, including access to serving railroad (unless shown in complete detail on site plans)

Figure 2-2 below is a typical CADD file showing a variety of different utility systems surveyed by Louisville District personnel at an Army Reserve Center. Each utility was coded in the field and placed on a separate MicroStation design file "level." Normally, as shown below, each utility system is given a different color coding as well as a different level assignment. Turning off different utility levels (and assigning these levels to separate sheets) eliminates the apparent overprinting below.

d. Detailed utility maps (sometimes referred to as Unit Layout Maps). At a scale of 1 inch = 100 ft or 1 inch = 50 ft and normally on reproductions of the detailed site plans, the detailed utility maps for large or complicated projects are generally prepared showing the details of all utilities on each sheet rather than with each utility on a separate sheet. By this means, the Facility Engineer is informed as to the underground relationship of the various utilities; and avoids the danger, in repairing one utility, of damaging another. Even at the relatively large scale of 1 inch = 50 ft, the complexity of utility data at "busy" intersections may require that inserts be added to the map at still larger scale. The detailed utility maps will show the additional detailed data of all kinds which it was not feasible to show on the smaller-scale general utility maps (grades at ground and inverts of manholes, etc., location and sizes of valves, service connections, etc.) and will thus provide the Facility Engineer with complete data necessary for operation and maintenance. On small projects where it has been feasible to prepare the general utility maps at large scale (1 inch = 50 ft), it will be possible to add the necessary complete detailed utility data to these maps and thus to avoid the preparation of special detailed utility maps. They may also be omitted in cases where it is found feasible to show utility details on "strip" road plans, providing the strips are of sufficient width to show service connections to structures on both sides of the road.

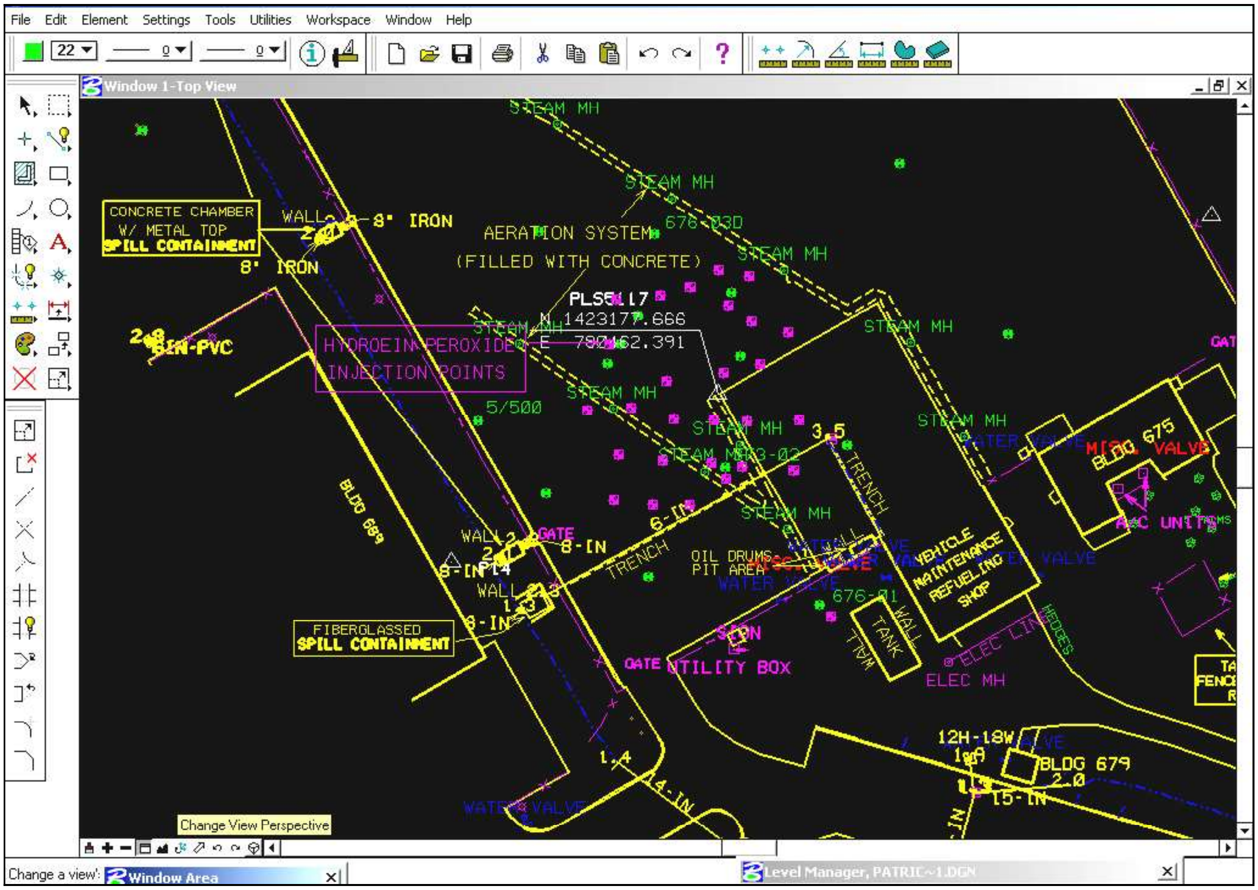


Figure 2-2. Utilities near a Vehicle Maintenance Refueling Facility at Patrick Air Force Base, Florida (Louisville District--2004)

2-7. As-Built Surveys

a. Purpose. As-built surveys are surveys compiled to show actual condition of completed projects as they exist for record purposes and/or payment. Since many field changes occur during construction, both authorized and clandestine, surveys are regularly completed to check the project against plans and specifications. As-built surveys are usually a modified version of the site plans that were originally required to plan the project. This is particularly true of road, railroad, or watercourse relocation projects. These projects are all of the route survey nature. The as-built survey, out of necessity, is this type also. Typical items checked are:

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

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

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

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

(2) Project monumentation is sometimes a requirement of as-built surveys since it is common to monument traverse lines and baselines for major projects. Their location should be checked for accuracy. In many areas it is common for such monumentation to be done by maintenance people who are not at all familiar with surveying and therefore the work is not always as accurate as would be desired.

2-8. Reservation Boundary Surveys and Maps

a. Purpose. Boundary surveys of civil and military reservations, posts, bases, etc. should generally be performed by a professional land surveyor licensed in the state where the work is performed. Corps of Engineers boundary survey requirements and procedures are detailed in Chapter 3 (Mapping) of ER 405-1-12 (*Real Estate Handbook*). This chapter in ER 405-1-12 describes the procedures for providing maps, surveys, legal descriptions, and related material on a project or installation basis for planning, acquisition, management, disposal, and audit of lands and interests in lands acquired by the Corps of Engineers for Department of the Army military and civil works projects, for the Department of the Air Force, and as agent for other Federal agencies. The criteria, general format, forms development, approval authority, maintenance, and distribution of project maps reflecting graphic depiction of all lands acquired and disposed of are described in this regulation. Boundary surveys include parcel maps, subdivision maps, plats, and any other surveys that are officially recorded (filed) in a local County or State clerk's office. The following are the important considerations regarding boundary surveys:

b. Method and accuracy. The determination of the position of all points should be by transit or total station traverses originating from and closing upon points of the local horizontal control system. The error of survey as indicated by any such closure before adjustment should not exceed the standards specified for boundary surveys in the particular state--usually ranging from 1 part in 5,000 to 1 part in 10,000. Where applicable, procedures and standards in the "*Manual of Instruction for the Survey of the Public Lands of the United States"* (US Bureau of Land Management 1973) should be followed.

c. Reconciliation of conflicting data. Cases occur in which the data supplied by the field surveys do not agree with the deed description; where these conflict and cannot be reconciled legal advice may be necessary. However, it is a generally-accepted rule that where discrepancies exist between the boundary line, as described in the deed, and the boundary lines of possession as defined on the ground by physical objects such as fences, marked trees, boundary stones, boundary roads, and streams, the physical evidence should be held as correct, unless convincing evidence to the contrary is produced.

d. Boundary monuments. When boundary surveys are authorized, standard reinforced concrete monuments should be placed at all angle points on the exterior boundary. In long courses, such monuments should be placed along the line of intervals of 1,000 ft on level terrain and at prominent points (intervisible if possible) where the terrain is rolling. Where a monument replaces an old landmark it should be clearly so noted in the notes of the survey and on the property map. The notes should fully describe old monuments or landmarks. Monumentation guidance is contained in EM 1110-1-1002 (*Survey Markers and Monumentation*) and in ER 405-1-12 (*Real Estate Handbook*).

e. Scales and other details. Boundary maps may be platted at any convenient scale consistent with the size of the area involved. In the case of small projects the scale of the site plan will be a convenient one for the boundary map. These maps should show the bearings, referred to the meridian of the Government control point to which the survey is tied, and the lengths of property lines and rights-of-way, the names of property owners, and the acreage to the nearest one-hundredth of an acre. Where survey bearings and lengths differ from the deed description the latter should also be shown. The maps should also show the position of all old property marks or corners that were found. The project coordinate grid lines, control points, and the coordinate values of the beginning corner of the survey (POB) should be shown on the maps.

f. Metes and bounds description. A complete typewritten metes and bounds description will be prepared and will include the bearings and distances of all courses of the exterior boundary of the reservation including detached areas, if such exist, with type of markers placed at angle and intermediate points. The description will state the acreage, the county and state within which the area is situated, and the date of the survey. The description should also include metes and bounds of any excepted area within the reservation such as roads, railroads, and easements, with areas thereof.

g. Reservation maps. Reservation maps are required wherever the entire area of a reservation, or portion thereof, is held in fee title by the Government. The scale of the reservation map will depend upon the area involved, reliable information available, and on the scale of reservation maps in existence. Until such time as the boundary surveying and monumenting of properties acquired by purchase or condemnation are authorized, reservation maps will be usually comprised of a compilation of all available existing information in the form of deed descriptions, General Land Office survey data, metes and bounds descriptions if available, etc. The following data will be clearly set forth on the reservation map:

- All exterior boundary lines (based on certified and monumented surveys if prepared).
- Total acreage (computed, if a survey has been authorized), together with acreage of lands transferred by temporary agreement, if any.
- The exterior boundaries and acreage of all additions to existing military areas acquired since preparation of earlier reservation maps--to be indicated on original maps of the reservation if practicable, otherwise on segmental maps properly coordinated with the original maps.
- The metes and bounds survey description, if previously authorized, or the General Land Office survey description, if available.
- The approximate outline and designation of all outstanding areas of operation which are a part of the military reservation; for example, Cantonment Areas, Ordnance Areas, Airfield and Revetment Areas, Hospital Areas, Maneuver Areas, Depot Areas, Radio Sites, Target Range Areas, Bombing Areas, and other areas of importance.
- Rights-of-way for sewerage, water supply lines, and other utility rights-of-way. If prepared on segmental maps, these should be properly coordinated with the reservation map.
- The State and County or Counties within which the reservation is located.

2-9. Reservoir Clearing Surveys

a. Purpose. Reservoir clearing surveys will include the establishment and mapping of the upper clearing limit, the marking of pertinent portions of dredging spoil areas, the location of clearing and grubbing limits for channel cut offs and improvements, and the lay out and marking of boat channels through areas which will not be cleared. The upper clearing limit comprises the major portion of the job, and work on this item should be vigorously prosecuted most particularly during the winter months while the foliage is off the trees. The clearing limit contour will be established on the ground, painted, staked, and mapped for all parts of the reservoir except such areas as may be prescribed to leave nucleated. The following are typical examples:

- Areas for the enhancement of game and fish habitat.
- Upstream limits of creeks where pool is confined within the old channel or within steep banks.
- Within the river banks except for designated sections where clearing in the vicinity of public areas will be performed.
- Islands formed by the clearing limit contour will not be cleared and must be located and marked. This will probably require some exploratory surveying in the wider areas of the reservoir.
- Flowage easements.

b. Contour establishment and mapping. The clearing limit may frequently be located in low, flat terrain with numerous branchings, "finger" sloughs, swales, and may lie in no pattern whatsoever. The prime objective of this type survey is to locate, paint, and map the clearing line contour. In running the levels to locate the contour, the rodmen will be the key to progress. The instrumentman will maintain a hand or electronic notebook, recording all readings for turns. The contour will be temporarily marked when located until a closure has been made and the notes verified. Mapping will be accomplished at a target scale of usually 1 inch = 500 ft.

c. Coverage. It is most probable that long traverses will be required in the lower reaches of the reservoir. In the large heavily wooded areas there will be numerous "finger" sloughs, islands, and peninsulas formed by the contour. Lines, at approximately 1 mile intervals, originating on control at the river banks should be run more or less normal to river, cross section style, a sufficient distance outward to encompass the clearing limit contour and then tied together to form loops. In running these lines, all points where the contour is crossed should be flagged and control points left for continuance of surveys. These lines would serve a dual purpose to locate areas under the clearing limit contour and, after adjustment, as control from which the surveys can be extended. Open areas will be delineated on the map, with proper annotation, such as "cultivated," "pasture," "pasture with scattered trees," "scattered brush and trees," or whatever, so that a classification of type clearing can be made. In large cleared areas the contour can be shot directly on aerial photos, if desired.

d. Markings. Trees standing on or very near the clearing line will be marked in accordance with specifications. These markings shall be placed at intervals such that they are readily intervisible at distances commensurate with the woods cover. All abrupt changes in direction shall be well marked at the point of turn and other prominent trees nearby in both directions. The contour will be marked in open areas--generally at 500 ft to 1000 ft intervals--or closer when required to show important changes in direction. Points where the marked contour is discontinued or resumed (such as the contour entering stream banks or a wildlife and fish habitat area where marking is not required) will be prominently marked. One or more trees should a marked, or enough to insure that this point is readily visible and identifiable.

e. Miscellaneous. Where no surveying of the contour is required within the riverbanks, careful watch for breakouts must be maintained. This may require leveling along the riverbanks and exploration of the mouths of the streams, ditches, sloughs to ensure that the contour is also confined and, therefore, information is needed from which to make a rigid estimate of the acreage involved. In the course of the survey the contour will be outlined and this should be sufficient to determine the length of each area. The width, from the contour down to the timberline next to the water's edge, must be obtained at proper intervals to furnish an excellent average width. Points where these widths are obtained and the widths shall be plotted.

2-10. Upland Disposal Area Surveys

Disposal areas are usually acquired in fee simple and thus will be surveyed and monumented in accordance with state or local codes and standards. Clearing of these areas will be on a graduated basis and it will be necessary under the clearing limit to part off only the portions of the areas required for the original dredging. Limits of the required areas will be furnished to the survey party, and the survey work can be accomplished using the area boundary monuments as control. These limits will be defined on the ground by appropriate markings. The clearing in the disposal areas will be of lesser quality than the reservoir clearing, so that clearing contour marking will prevail and the contour should be marked in the spoil areas as elsewhere.

2-11. Channel Improvement and Cutoff Surveys

These areas require modified grubbing in addition to clearing and will have to be delineated differently from the normal reservoir clearing. The clearing and grubbing lines for these features are usually outlined in detail on the hydrographic maps and the cut off topographic maps. Using the existing ground control and these layouts, these limits will be established. All angle points in these limits will be marked on the nearest tree. Enough intervening trees to promote intervisibility will be marked. In order to furnish data for a close estimate of area of the channel improvement sites, the streamward edge of the timber shall be shown. Open areas within the cutoff confines will also be shown. This clearing will be of a higher grade and take precedence over the regular reservoir clearing. If it is found that the clearing contour passes through any other channel improvement clearing areas, and at the cutoff areas, marking should be discontinued at the intersection. No marking of the clearing limit contour will be done in the areas of clearing for channel improvement. Clearing maps will be presented as line drawn maps, using the reservoir mapping as a base and utilizing the cultural data, stream outlines, etc.

2-12. Post-Flood High Water Mark Surveys

High water marks are evidence of the highest stages reached by a flood. There are many different types of marks and the proper identification of them is the key to getting meaningful data. For this reason the most experienced man in the field party shall be used to locate the marks. High water marks tend to disappear rapidly after the flood peak. Thus, the survey should be started as soon as possible after the peak. Marks should be identified by means of stakes, flagging, paint, nails, crayon, etc., and field sketches made of locations to guide future survey work. Elevations can be obtained for the marks when time permits. Criteria for selecting high water marks are as follows:

- Marks should be selected on surfaces parallel to the line of flow so that they represent the water surface and not the energy grade line of the stream.
- Often small seeds of various plants will provide excellent high water marks, remaining in the crevices of bark or in cracks in fence posts or utility poles. The highest of such particles should be used.

- Mud or silt will often leave easily recognizable lines along banks or on trees, brush, rocks, buildings, etc.
- Excellent high water marks may be found on buildings within the flood plane. Excellent marks may be found on windowpanes or screens. However, care must be exercised not to select marks on the upstream side of the building, which may have been affected by velocity head.
- Residents may afford a valuable source of information when evidence has been cleaned up or destroyed by rain. Such information may be particularly reliable where the water has come into buildings on the premises.

Poorly defined high water marks:

- Drift found on bushes or trees in or near the channel may afford false information. On trees the buildup on the upstream side caused by current may cause an abnormally high reading, and conversely on the downstream side. Bushes may bend with the current and, after the slowdown, straighten up to lift the drift above the peak flood elevation.
- Foam lines, commonly found on bridge abutments, wingwalls, riprap, poles, trees, etc., may be affected by velocity head pileup.
- Wash lines are usually poor.
- Information from residents after passage of time and destruction of evidence, especially when remote from place of residence.

The type of marks should be fully described to facilitate recovery by others. Examples include "drift on bank," "drift on tree," "wash line," "silt line on post," etc. Marks should be rated as "excellent," "good," "fair," or "poor." Attribute data may include:

- Number, river or basin, bank of river.
- Month and year of flood.
- State, county, nearest town.
- Section, Township, Range, if possible.
- Specific location by description from nearest town, etc.
- Specific description of mark, e.g., "nail in post."
- Source of information.
- Type of mark.
- Sketch on right hand page of notebook.

If possible, mark may be tied to existing horizontal control. Otherwise, autonomous/differential GPS or spotting on quadrangle or other map may be the only available method of locating, especially in remote areas. All marks should be leveled to Third Order accuracy standards. Readings should be to nearest 0.01 ft. A TBM should be set in the vicinity to facilitate ties to any future comparative marks.

2-13. Route Surveys

Route surveys are most commonly used for levees, stream channels, highways, railways, canals, power transmission lines, pipelines, and other utilities. In general, route surveys consist of:

- Determining ground configuration and the location of objects within and along a proposed route.
- Establishing the linear or curvilinear alignment of the route.
- Determining volumes of earthwork required for construction.

After the initial staking of the alignment has been closed through a set of primary control points and adjustments have been made, center-line/baseline stationing will identify all points established on the route. Differential levels are established through the area from two benchmarks previously established. Cross-sections in the past were taken left and right of centerline. Today digital terrain models (DTM) or photogrammetry is used to produce cross-sections for design grades. Surveys may be conducted to check these sections at intermittent stations along the centerline. Ground elevations and features will be recorded as required.

2-14. Bridge Surveys

Bridge surveys are often required for navigation projects. In many instances, a plan of the bridge may be available from the highway department, county engineer, railroad, etc. When as-built drawings can be obtained, it may be substituted for portions of the data required herein. However, an accuracy check should be made in the field. Field survey measurements should include the elevation of bridge floor, low steel, and a ground section under bridge to present an accurate portrait of the bridge opening. Piers, bents, and piling should be located with widths or thicknesses being measured so that their volume can be computed and deducted from the effective opening under the bridge. Superstructure is not important and need not be shown. However, guardrail elevations should be obtained and the rails described as to whether they are of solid or open construction. A very brief description of bridge as to type of construction (wood, steel, concrete, etc.) and its general condition should be furnished. If wingwalls exist, minimum measurements should be made to draw them in proper perspective. Sketches of existing bridges may be required on many of the various types of surveys performed by a District. Obtain digital photographs of all bridges. All photographs should be carefully indexed, and a sketch made to show approximate position and angle of each exposure. In general, survey data to be included on bridge survey field notes should include:

- Direction facing bridge, whether upstream or downstream.
- Length of bridge, or stationing if established.
- Distances from center to center of piers or bents.
- Dimensions of piers or pilings including batter.
- Low chord (or steel) elevation over channel.
- Profile of bridge deck, roadway, handrail, etc.
- Sketch of plan of bridge, when required, showing deck dimensions including girder size and spacing.
- Sketch of typical bent, when required, including cap size.
- Sketch of wingwall, when required.
- Digital photographs from various aspects.
- Material of which constructed (wood, steel, concrete, prestressed concrete, etc.).
- Type of construction, such as truss, trestle, or girder.
- General condition of bridge.
- Alignment of bridge to channel, whether normal or at angle (may be shown on plan sketch).
- Alignment of piers or bents, whether normal or at angle.
- Composition of bents may be indicated on sketch.
- Designation of highway, road, street, railroad, etc., utilizing bridge--describe surface, if road.

An example of field notes for a typical bridge survey is shown in Chapter 12.

2-15. Artillery Surveys (FM 3-34.331)

The Field Artillery (FA) is a primary user of precise positioning and orientation information in a wartime environment. Topographic survey support must be provided to multiple launch rocket system (MLRS) units, Corps's general support units, and other nondivisional assets in the Corps area. The FA requires that topographic surveyors:

- Establish and recover monumented survey control points (SCPs)--horizontal and vertical--and azimuthal references for conventional and inertial FA survey teams.
- Coordinate the exact position of the high-order control with the Corps's survey officer.
- Augment FA survey sections when appropriate.

Topographic-engineer companies are the primary source of topographic support throughout the Echelons above Corps (EAC) and general support. Topographic companies support artillery surveys by:

- Extending horizontal and vertical control into the corps and division areas.
- Providing a survey planning and coordination element (SPCE) in support of the EAC.
- Providing mapping-survey control where required.
- Advising on topographic matters.
- Assisting in lower-level surveys to augment FA surveys.

Additional details on field artillery surveys are found in Chapter 9 of FM 3-34.331

2-16. Airport Obstruction and NAVAID Surveys (FM 3-34.331)

Airfield-obstruction and NAVAID surveying operations involve obtaining accurate and complete NAVAID (and associated airport/heliport-obstruction) and geodetic positioning data. Airport obstruction chart (AOC) surveys provide source information on:

- Runways and stopways.
- NAVAIDs.
- Federal Aviation Regulation defined obstructions.
- Aircraft-movement aprons.
- Prominent airport buildings.
- Selected roads and other traverse ways.
- Cultural and natural features of landmark value.
- Miscellaneous and special-request items.

a. NSRS connection requirements. AOC surveys also establish or verify geodetic control in the airport vicinity is accurately connected to the NSRS. This control and the NSRS connection ensure accurate relativity between these points on the airport and other surveyed points in the NSRS, including GPS navigational satellites. AOC data is used to:

- Develop instrument approach and departure procedures.
- Determine maximum takeoff weights.
- Certify airports for certain types of operations.
- Update official aeronautical publications.

- Provide geodetic control for engineering projects related to runway/taxiway construction, NAVAID positioning, obstruction clearing, and other airport improvements.
- Assist in airport planning and land-use studies.
- Support activities such as aircraft accident investigations and special projects.

b. Survey standards. Federal Aviation Administration Publication 405 (*Standards for Aeronautical Surveys and Related Products*, Fourth Edition., 1996.) and Federal Aviation Regulation 77 (*Objects Affecting Navigable Airspace*, 15 July 1996) outline the requirements for AOC surveys. Various areas, surfaces, reference points, dimensions, and specifications used in airfield surveys are described in these references.

c. Runway surveys. All length and width measurements are determined to the nearest foot. If the runway's threshold is displaced, the distance (in feet) is given from the beginning of the runway's surface. Determine the coordinates (latitude and longitude) of the runway's threshold and stop end at the runway's centerline. Elevations at the runway's threshold, stop end, and highest elevation (within the first 3,000 feet of each runway touchdown zone elevation [TDZE]) should be determined to the nearest 0.1 ft from the MSL. In addition, prepare runway profiles that show the elevations listed above, the runway's high and low points, grade changes, and gradients. Determine the elevation of a point on the instrumented runway's centerline nearest to the instrument landing system (ILS) and the glide-path transmitter to the nearest 0.1-ft MSL.

d. NAVAID surveys. Airports requiring airfield-obstruction and NAVAID surveys are instrumented runways. The exact point on the radar, the reflectors, the runway intercepts, and the instrument landing system (ILS) and microwave-landing-system (MLS) components depend on the survey type, the location, and the required accuracy. The requirement to verify the existing ILS/MLS, their proper description, and all components on or near the runway are mandatory. Obtain information for locating and describing all airfield features with help from airfield operation and maintenance section, and control tower personnel.

e. Obstruction surveys. An obstruction is an object or feature protruding through or above any navigational imaginary surface that poses a threat to the safe operation of aircraft. Navigational imaginary surfaces or obstruction identification surfaces are defined in Federal Aviation Regulation 77.

f. Reference. Additional details (including accuracy specifications) on airport obstruction and NAVAID surveys are found in Chapter 10 of FM 3-34.331

Figure 2-3. Airfield NAVAID positioning using Fast-Static GPS methods

2-17. Site Plan Engineering Drawings

An engineering site plan survey is a topographic (and, if necessary, hydrographic) survey from which a project is conceived, justified, designed, and built. Types of surveys that are performed for developing site plan drawings include:

- Hydrographic Surveys--surveys of USACE navigation, flood control, or reservoir projects (see EM 1110-2-1003 (*Hydrographic Surveying*).
- Beach Profile Surveys--surveys of renourishment projects, shore protection features, and structures.
- Levee Surveys--surveys of levees, revetments, and other related river control structures.
- Route Surveys--surveys of proposed or existing transportation routes.
- Quantity Surveys--surveys for construction measurement and payment.
- Structure Surveys--surveys of facilities, utilities, or structures.

The methods used in performing an engineering survey can and sometimes will involve all of the equipment and techniques available. GPS survey techniques are covered in EM 1110-1-1003 (*NAVSTAR GPS Surveying*). Many of these GPS techniques are used in establishing control for topographic surveys. Photogrammetry may also be used to produce maps of almost any scale and corresponding contour interval--see EM 1110-1-1000 (*Photogrammetric Mapping*). Profiles and cross sections may also be obtained from aerial photos. The accuracy of the photogrammetric product varies directly with the flight altitude or photo scale--these factors must be considered when planning such a project. Surveys of structural deformations are not topographic surveys. These types of surveys are detailed in EM 1110-2-1009 (*Structural Deformation Surveying*). Preliminary, General, and Detailed Site Plans are often specified, as described below.

a. Preliminary Site Plans (Pre-engineering surveys). Based on reconnaissance surveys previously described, the first plans to be prepared are preliminary site studies showing in skeleton form only the general arrangement of areas where construction will take place, circulation between them and to training areas, public roads, and serving railroads. The most convenient scale for these studies on sizeable projects has usually been 1 inch = 400 ft; on smaller projects, 1 inch = 300 or 200 ft.

b. Approved General Site Plan. Upon the basis of these studies, a general site plan, at the same scale, is reached; and serves as basis for enlargement for detailed site plans and for general utility plans described below. This approved site plan will show:

- Grid System.
- Buildings (indicate types).
- Wooded Areas
- Roads and Fences (indicate roadway widths and types of base and surface).
- Service and Parking Areas.
- Rail Facilities, including access to serving railroad.
- Use Areas of all kinds (runways, aprons, firebreaks, parade grounds, etc.)

c. Detailed Site Plans. Based on the topographic surveys described above, and enlarged from the approved preliminary site plan, detailed site plans at the scale of 1 inch = 200 ft or larger are prepared. In the case of large developments such as cantonments, the detailed site plans often encompass an area about equal to that utilized by a regiment and may be drawn at as large a scale as 1 inch = 50 ft. The detailed site plans serve as a basis for detailed utility plans.

2-18. Army Installation Mapping Requirements

Each installation is guided by its respective service's comprehensive or master planning requirements. Each installation, depending on its mission, may have substantially more or fewer theme specific maps. It is the responsibility of the installation's planning, environmental operations, engineering, and administrative staff to understand the mapping needs for their installation. Each installation is unique and the specific quantity and type of maps required for an installation depend upon its individual features, conditions, and requirements. An installation will generally produce and maintain a set of maps to meet both its planning and operational needs. The following list (from ERDC/ITL 1999b--*CADD/GIS Technology Center Guidelines for Installation Mapping and Geospatial Data*) is representative of the geospatial map layers needed on a typical installation.

A-Natural and Cultural Resources
A-1 Areas of Critical Concern
- Historic Preservation and Archeology
- Threatened and Endangered Species
- Wetlands & Floodplains
- State Coastal Zones
- Lakes, Rivers, Streams, and Water Bodies
- Soil Borings & Soil Types

A-2 Management Areas
- Geology, including Surface Features
- Topography & Physiology
- Hydrology
- Vegetation Types
- Forest (Commercial Timber)
- Agriculture Grazing/Crops
- Fish and Wildlife
- Prime & Unique Soils
- Grounds Categories
- Climate & Weather
- Bird Aircraft Strike Hazard (BASH)
- Outdoor Recreation
- Pest Management

B-Environmental Quality
B-1 Environmental Regulatory
- Hazardous Waste Generation Points
- Permitted Hazardous Facilities
- Solid Waste Generation Points
- Solid Waste Disposal Locations
- Fuel Storage Tanks
- Installation Restoration Program (IRP)

B-2 Environmental Emissions
- Air Emission
- Waste Water NPDES Discharge
- Storm Water NPDES Discharge
- Drinking Water Supply Sources
- Electromagnetic and Radiation Sources
- Radon Sources

C-Layout and Vicinity Maps
C-1 Installation Layout
C-2 Off-base Sites
C-3 Regional Location
C-4 Vicinity Location
C-5 Aerial Photographs
C-6 Installation Boundary

D-Land Use
D-1 Existing Land Use
D-1.1 Future Land Use
D-2 Off-base Sites Land Use
D-2.1 Off-base Sites Future Land Use
D-3 Real Estate
D 4 Explosive Safety Quantity-Distance (QD) Arc
D-5 Hazard Analysis Constraints
D-6 Composite Installation Constraints and Opportunities
D-7 Area Development

E-Airfield Operations
E-1 On base Obstruction to Airfield and Airspace Criteria
E-2 Approach and Departure - Zone Obstructions to 10,000 Ft
E-3 Approach and Departure Zone Obstructions Beyond 10,000 Ft
E-4 Airspace Obstruction - Vicinity
E-5 Terminal Enroute Procedures (TERPS) Automation Plan
E-6 Airfield and Airspace Clearances
- Waivers
- Clear Zones
- Primary Surfaces
- Transitional Surface (7:1)
- Approach &Departure Surface (50:1)
- Approach and Taxiway Clearances

E-7 Airfield Pavement Plan
E-8 Airfield Pavement Details
E-9 Aircraft Parking Plan
E-9.1 Proposed Aircraft Parking Plan
E-10 Airfield Lighting Systems

F- Reserved
F-1 Reserved
F-2 Reserved

G-Utilities System Plan
G-1 Water Supply System
G-2 Sanitary Sewerage System
G-3 Storm Drainage System
G-4 Electrical Distribution System (Street & Airfield)
G-5 Central Heating and Cooling System
G-6 Natural Gas Distribution System
G-7 Liquid Fuel System
G-8 Cathodic Protection System
G-9 Cathodic Protection System Details
G-10 Industrial Waste and Drain System
G-11 Composite Utility System Constraints
G-11.1 Central Aircraft Support System
G-12 Other Utility Systems

H-Communication and NAVAID Systems
H-1 Installation Communication (Base and civilian communications units)
H-2 NAVAIDs and Weather Facilities

I-Transportation System
I-1 Community Network Access to Base
I-2 On-base Network
I-2.1 Future Transportation Plan

J-Energy Plan

K-Architectural Compatibility

L-Landscape Development Area

M-Future Development
M-1 Current Status
M-2 Short-Range Development
M-3 Long-Range Development

N-Reserved
N-1 Reserved
N-2 Reserved

O- Force Protection
O-1 Surge Capability (Beddown and Support of Deployed Forces)
O-2 Physical Security
O-3 Disaster Preparedness Crash Grid Map
O-4 Air Base Survivability and Theater-Specific Requirements

P - Ports and Harbors

R - Range and Training Areas

The CADD/GIS Technology Center schema (i.e., database structure/format) for installation maps uses "entity sets" to classify graphic (maps) and non-graphic data (tabular files, reports, database files. etc.). The overall structure is based upon the concept of features, attributes, and values. There are 26 entity sets listed in the latest release of the Spatial Data Standard for Facilities, Infrastructure, and Environment (SDSFIE):

Auditory
Boundary
Buildings
Cadastre
Climate
Common
Communications
Cultural
Demographics
Ecology
Environmental Hazards
Fauna
Flora
Future Projects
Geodetic
Geology
Hydrography
Improvement
Land Status
Landform
Military Operations
Olfactory
Soil
Transportation
Utilities
Visual

These 26 entity sets are further broken down into Entity Class, Entity Type, and Entity (attributes, descriptors, ranges, etc.), as shown for a typical feature in the plate below.

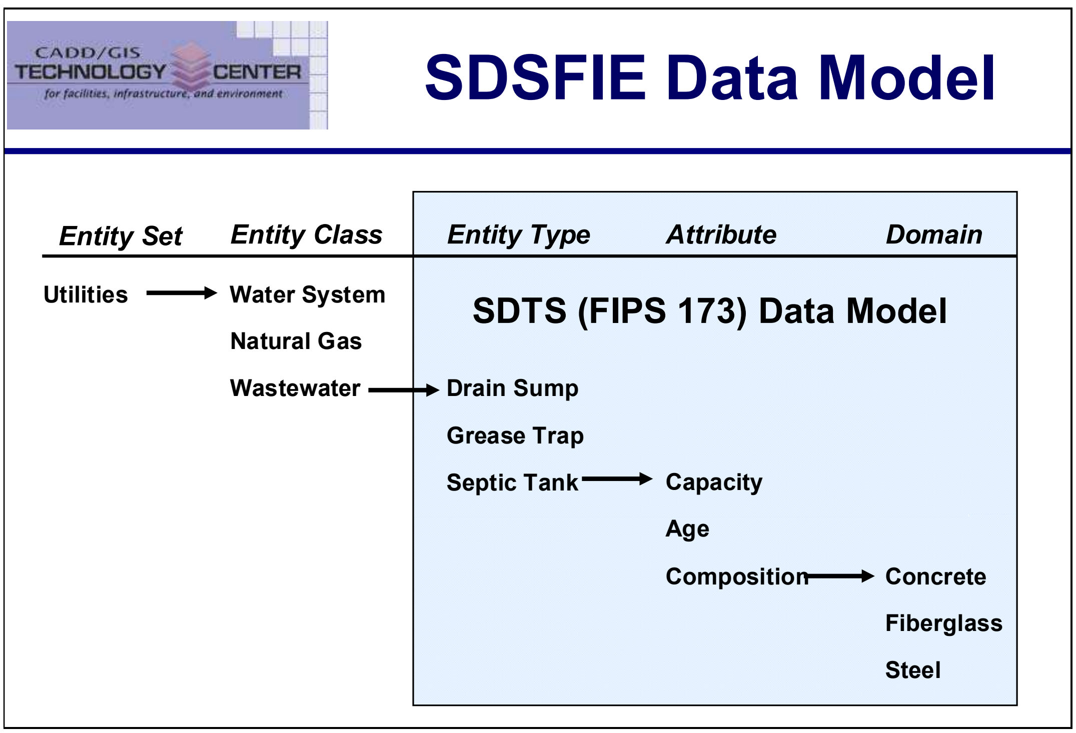


Figure 2-4. Spatial data feature and attribute breakdown for a septic tank (SDSFIE Release 2.300)

Maps supporting Army installations are expected to conform to the SDSFIE data model, and the related Facility Management Standard for Facilities, Infrastructure, and Environment (FMSFIE) non-graphic tables and elements.

SECTION II
Survey Methods and Techniques

This section provides an overview of the past and present instruments and methods used to perform topographic surveys of sites, facilities, or infrastructure.

2-19. Older Topographic Surveying Methods

Prior to the advent of total stations, GPS, LIDAR, and data collector systems, transit and plane table topographic surveying methods and instruments were once standard. They are rarely used today, other than perhaps for small surveys when a total station or RTK system is not available. However, the basic field considerations regarding detail and accuracy have not changed, and field observing methods with total stations or RTK are not significantly different from the older survey techniques briefly described in the following sections.

a. Transit-Tape (Chain). Transit tape topographic surveys can be used to locate points from which a map may be drawn. The method generally requires that all observed data be recorded in a field book and the map plotted in the office. Angles from a known station are measured from another known station or azimuth mark to the point to be located and the distance taped (or chained) from the instrument to the point. Transit-tape surveys typically set a baseline along which cross-section hubs were occupied and topographic features were shot in on each cross-section. The elevation of an offset point on a section is determined by vertical angle observations from the transit. The slope or horizontal distance to the offset point is obtained by chaining. The accuracy may be slightly better than the plane table-alidade method or very high (0.1 ft or less), depending upon the equipment combinations used. Transits are still used by some surveying and engineering firms, although on a declining basis if electronic total station equipment is available. Transit-tape surveys can be used for small jobs, such as staking out recreational fields, simple residential lot (mortgage) surveys, and aligning and setting grade for small construction projects. Assuming the project is small and an experienced operator is available, this type of survey method can be effective if no alternative positioning method is available. Detailed procedures for performing and recording transit tape topographic surveys can be found in most of the referenced survey texts listed at Section A-2.

b. Chaining. 100, 200, 300, or 500-foot steel tapes are used for manual distance measurement methods. Woven, cloth, and other types of tapes may also be used for lower accuracy measurements. Maintaining any level of accuracy (e.g., better than 1: 5,000) with a steel tape is a difficult process, and requires two experienced persons. Mistakes/blunders are common. Tapes must be accurately aligned over the points (using plumb bobs), held at a constant, measured tension, and held horizontally (using hand levels). Subsequent corrections for tape sag/tension, temperature, and slope may be necessary if a higher accuracy is required. Taping methods, errors, and corrections are not covered in this manual but may be found in any of the basic surveying texts listed at Appendix A-2.

c. Transit-Stadia. Transit-stadia topographic surveys are performed similarly to transit-tape surveys described above. The only difference is that distances to offset topographic points are measured by stadia "tacheometry" means-- i.e., using the distance proportionate ratio of the horizontal cross hairs in the transit telescope. The multiple horizontal crosshairs in the transit scope can be used to determine distance when observations are made on a level rod at the remote point. This distance measurement technique has been used for decades, and is also the basis of plane table survey distance measurement. The three horizontal crosshairs in the transit are spaced such that the upper and lower crosshair will read 1.0 ft on a rod 100 ft distant from the transit--a "stadia constant" ratio of 100 : 1. (Not all instruments have an even 100 : 1 stadia constant). The accuracy of a stadia-derived distance is not good--probably about 1 : 500 at best. Thus, a 500 ft shot could have an error of ± 1 ft. Additional errors (and corrections) result from inclined stadia measurements, i.e., when the shot is not horizontal. Reduction of the stadia intercept values to a nominal slope distance, then reduction to horizontal, requires significant computation or use of tables. Transit-stadia was often used like a modern day total station in that topo detail could be densified (typically using radial survey methods) from a single instrument setup. All observed data was recorded in a field book, and occasionally optionally plotted in the field. Transit-stadia techniques are likewise rarely performed today if a total station is available. Details on stadia measurement methods are found in any surveying textbook--e.g., Kavanagh 1997.

d. Transit/Theodolite-EDM. Electronic Distance Measurement (EDM) instruments were first developed in the 1950s, primarily for geodetic operations. In the 1970s, more compact EDM units were mounted atop or alongside transits and theodolites--thus replacing manual chaining or optical stadia distance measurement. Observed data were still recorded in field books for later office hand plotting. These crude transit-EDM combinations were the early forerunner of the modern total stations. During this time, methods were developed for automated drafting of observed features--after individual angles and distances and features were encoded on punch cards and input to a computer/plotter system.

Figure 2-5. Plane table and alidade--Wild T-2 theodolite at right (USC&GS, ca 1960s)

e. Plane table surveying. The plane table and alidade were once the most common tools used to produce detailed site plan maps in the field. The Egyptians are said to have been the first to use a plane table to make large-scale accurate survey maps to represent natural features and man-made structures. Plane table mapping is rarely done today--plane table surveying has, for most purposes, been replaced by aerial photogrammetry and total stations, but the final map is still similar. Plane table surveys were performed in the Corps well into the 1980s, and perhaps into the 1990s in some districts. A plane table survey system is described as follows: A blank map upon which control points and grid ticks have been plotted is mounted on the plane table. The table is mounted on a low tripod with a specially made head--see Figure 2-5 above. The head swivels so that it can be leveled, locked in the level position, and then be rotated so that the base map can be oriented. The base map is a scaled plot of the ground control stations.

Thus, with the table set up over one of the stations, it can be rotated so that the plotted stations lie in their true orientation relative to the points on the ground. Spot elevations and located features are located with an alidade, an instrument that uses optical stadia to determine distance (similar to the transit stadia). The error of a map produced with a plane table and alidade varies across the map as the error in stadia measurements varies with distance. Horizontal errors may range from 0.2 ft at 300 feet, to 10 ft or more at 1,000 feet. Since the elevation of the point is determined from the stadia measurement, relative errors in the vertical result. The plane table survey resulted in a “field-finished” map product, with all quality control and quality assurance performed in the field by the party chief/surveyor. The site plan map delivered from the plane table was immediately suitable for overlaying design detail. Modern day electronic survey and CADD systems are still attempting to attain the same level of “field-finish” capability that the plane table once produced. Older editions of this manual or the surveying textbooks listed in Section A-2 should be consulted if more detailed information on plane table survey techniques and alidade observations is needed.

Figure 2-6. Leica TCR 705 Reflectorless Total Station

2-20. Total Stations

Total stations were first developed in the 1980s by Hewlett-Packard (Brinker and Minnick 1995). These instruments sensed horizontal and vertical angles electronically instead of optically, and combined them with an EDM slope distance to output the X-Y-Z coordinates of a point relative to the instrument's X-Y-Z coordinates. Electronic theodolites operate in a manner similar to optical instruments. Angle readings can be to 1" with precision to 0.5". Digital readouts eliminate the uncertainty associated with reading and interpolating scale and micrometer data. The electronic angle-measurement system eliminates the horizontal- and vertical-angle errors that normally occur in conventional theodolites. Measurements are based on reading an integrated signal over the surface of the electronic device that produces a mean angular value and eliminates the inaccuracies from eccentricity and circle graduation. These instruments also are equipped with a dual-axis compensator, which automatically corrects both horizontal and vertical angles for any deviation in the plumb line. An EDM device is added to the theodolite and allows for the simultaneous measurements of the angle and the distance. With the addition of a data collector, the total station interfaces directly with onboard microprocessors, external PCs, and software. The ability to perform all measurements and to record the data with a single device has revolutionized surveying. Total stations perform the following basic functions:

Types of measurements:

- Slope distance
- Horizontal angle
- Vertical angle

Operator input to total station data collector:

- Text (date, job number, crew, etc.)
- Atmospheric corrections (PPM)
- Geodetic/grid definitions
- HI & HR
- Descriptor/attribute of setup point, backsight point, sideshot point, stakeout point, etc

In general, there are three types of total station operating modes:

- Reflector--total station requires a solid reflector or retroreflector signal return from the remote point to resolve digital angles and distances. Prisms are attached to a pole positioned over a feature. Requires two-man field crew--operator and rodman.

- Reflectorless--the total station will resolve (and coordinate) signal returns off natural features. Distances may be far more limited than those obtained from reflectors ... typically less than 1,000 ft. Allows for more economical one-man field crew operation.

- Robotic--total station self-tracks single operator/rodman at remote shot or stakeout points. One-man crew operation, with operator normally based at remote rod point.

Additional details on total stations are covered in Chapter 8. Total stations are also extensively covered in the text references listed at Appendix A-2--e.g., Wolf and Ghilani 2002.

Figure 2-7. RTK base station and radio link transmitter--and rover with backpack (Key West Harbor Dredging Project 2004--C&C Technologies, Inc. & Jacksonville District)

2-21. Real Time Kinematic (RTK) GPS

RTK survey methods have become widely used for accurate engineering and construction surveys, including topographic site plan mapping, construction stake out, construction equipment location, and hydrographic surveying. RTK survey systems operate in a similar fashion as the robotic total station, with one major exception being that a visual line of sight between the reference point and remote data collection point is not required. Both RTK and total stations use similar data collection routines and methods, and can perform identical COGO stake out functions. Kinematic surveying is a GPS carrier phase surveying technique that allows the user to rapidly and accurately measure baselines while moving from one point to the next, stopping only briefly at the unknown points, or in dynamic motion such as a survey boat or aircraft. A reference receiver is set up at a known station and a remote, or rover, receiver traverses between the unknown points to be positioned. The data is collected and processed (either in real-time or post-time) to obtain accurate positions to the centimeter level. Real-time kinematic solutions of X-Y-Z locations using the carrier (not code) phase are referred to as "real-time kinematic" (RTK) surveys. However, included in this definition are "post-processed real-time kinematic" (PPRTK) techniques where the kinematic solution is not actually performed in "real-time." RTK (or PPRTK) survey techniques require some form of initialization to resolve the carrier phase ambiguities. This is done in real-time using "On-the-Fly" (OTF) processing techniques. Periodic loss of satellite lock can be tolerated and no static initialization is required to regain the integers. This differs from other GPS techniques that require static initialization while the user is stationary. A communication link between the reference and rover receivers is required to maintain a real-time solution. Additional details on performing topographic surveys with RTK are covered in Chapter 9.

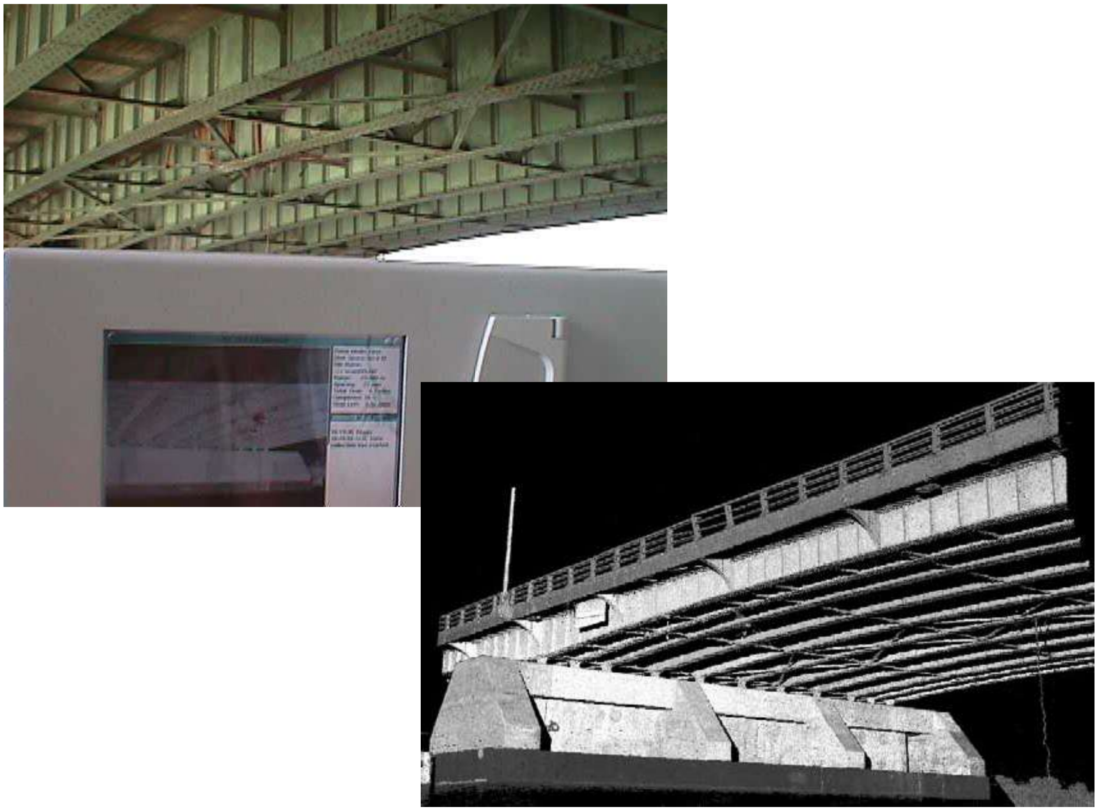


Figure 2-8. Optech LIDAR scanner and resultant image of underside of Kennedy Bridge (Arc Surveying & Mapping, Inc.--Puerto Rico 2002)

2-22. Terrestrial LIDAR (Laser) Scanning

Laser scanning instruments have been developed that will provide topographic detail of structures and facilities at an extremely high density, as shown in Figure 2-8 above. These tripod-mounted instruments operate similarly to a reflectorless total station. However, they are capable of scanning the entire field of view with centimeter-level pixel density in some cases. A full 3D model of a project site or facility results from the scan. This model must be edited and feature attributes added. See Chapter 10 for additional descriptions of terrestrial LIDAR survey procedures and applications.

2-23. Topographic Data Collection Procedures

Uniform operating procedures are needed to avoid confusion when collecting topographic survey data, especially for detailed utility surveys. The use of proper field procedures is essential to prevent confusion in generating the final site plan map. Collection of survey points in a meaningful pattern aids in identifying map features. The following guidelines are applicable to all types of topographic survey methods, including total stations and RTK systems.

a. Establish primary horizontal and vertical control for radial survey. This includes bringing control into the site and establishing setup points for the radial survey. Primary control is usually brought into the site from established NSRS monuments/benchmarks using static or kinematic GPS survey methods and/or differential leveling. Supplemental traverses between radial setup points can be conducted with a total station as the radial survey is being performed. A RTK system may require only one setup base; however, supplemental checkpoints may be required for site calibration. Elevations are established for the radial traverse points and/or RTK calibration points using conventional leveling techniques. Total station trigonometric elevations or RTK elevations may be used if vertical accuracy is not critical--i.e., ± 0.1 ft.

b. Perform radial surveys to obtain information for mapping. Set the total station or RTK base over control points established as described above. Measure and record the distance from the control point up to the electronic center of the instrument (HI), as well as the height of the prism or RTK antenna on the prism pole (HR). To prevent significant errors in the elevations, the surveyor must report and record any change in the height of the prism pole. For accuracy, use a suitable prism and target that matches optical and electrical offsets of the total station. Use of fixed-height (e.g., 2-meter) prism poles is recommended for total station or RTK observations, where practical.

c. Collect topographic feature data in a specific sequence. Collect planimetric features (roads, buildings, etc.) first. Enter ground elevation data points needed to fully define the topography. Observe and define break lines. Use the break lines in the process of interpolating the contours to establish regions for each interpolation set. Contour interpolation will not cross break lines. Assume that features such as road edges or streams are break lines. They do not need to be redefined. Enter any additional definition of ridges, vertical, fault lines, and other features.

d. Draw a sketch of planimetric features. A field book sketch or video of planimetric features is an essential ingredient to proper deciphering of field data. The sketch may also be made on a pen tablet PC. The sketch does not need to be drawn to scale and may be crude, but must be complete. Numbers listed on the sketch show point locations. The sketch helps the CADD operator who has probably never been to the jobsite confirm that the feature codes are correct by checking the sketch.

e. Obtain points in sequence. The translation of field data to a CADD program will connect points that have codes associated with linear features (such as the edge of road) if the points are obtained in sequence. For example, the surveyor should define an edge of a road by giving shots at intervals on one setup. Another point code, such as natural ground, will break the sequence and will stop formation of a line on the subsequent CADD file. The surveyor should then obtain the opposite road edge. Data collector software with "field-finish" capabilities will facilitate coding of continuous features.

f. Use proper collection techniques. Using proper techniques to collect planimetric features can give automatic definition of many of these features in the CADD design file. This basic picture helps in operation orientation and results in easier completion of the features on the map. Improper techniques can create problems for office personnel during analysis of the collected data. The function performed by the surveyor in determining which points to obtain and the order in which they are gathered is crucial. This task is often done by the party chief. Cross training in office procedures gives field personnel a better understanding of proper field techniques.

(1) Most crews will make and record 250 to over 1,000 measurements per day, depending on the shot point detail required. This includes any notes that must be put into the system to define what was measured. A learning curve is involved in the establishment of productivity standards. A crew usually has to complete five to six mapping projects to become confident enough with their equipment and the feature coding system to start reaching system potential.

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

measurement and another 5-10 seconds coding the measurement. The same time sequences are applicable for a one-man topographic survey using a robotic total station or RTK.

(3) When the general spacing of the measurements exceeds 50 feet, having a second rod person may increase productivity. A second rod person allows the crew to have a target available for measurement when the instrument operator is ready to start another measurement coding sequence. Once the measurement is completed, the rod person can move to the next shot, and the instrument operator can code the measurement while the rod people are moving. If the distance of that move is 50 feet or greater, the instrument will be idle if you have only one rod person.

(4) Communication between rod person and instrument person is commonly done via radio or cell phone. The rodmen can work independently in taking ground shots or single features; or they can work together by leapfrogging along planimetric or topographic feature lines. When more than one rod person is used, crew members should switch jobs throughout the day. This helps to eliminate fatigue in the person operating the instrument.

2-24. Automated Field Data Collection

Since the 1990s, survey data collection has progressed from hand recording to field-finish data processing. Prior to the implementation of data collectors, control survey data and topographic feature data were recorded in a standard field book for subsequent office adjustment, processing, and plotting. Modern data collectors can perform all these functions in the field. This includes least squares adjustments of control networks, full feature attributing, symbology assignment to features, and on-screen drafting/plotting capabilities. Data collectors either are built into a total station or are separate instruments. A separate (independent) data collector is advantageous in that it can be used for a variety of survey instruments--e.g., total station, digital level, GPS receiver. Field data collector files are downloaded to an office PC platform where the field data can be edited and modified so it can be directly input into a CADD or GIS software package for subsequent design and analysis uses. Many upgraded CADD/GIS software packages can directly download field data from the collector without going through interim software (e.g., CVTPC). Subsequent chapters in this manual (i.e. Chapters 7 and 11) provide additional information on data collectors and the transition of field collected data to office processing systems.

a. Field survey books. Even with fully automated data collection, field survey books are not obsolete. They must be used as a legal record of the survey, even though most of the observational data is referenced in a data file. Field books are used to certify work performed on a project (personnel, date, time, etc.). They are also necessary to record detailed sketches of facilities, utilities, or other features that cannot be easily developed (or sketched) in a data collector. When legal boundary surveys are performed that involve ties to corners, it is recommended that supplemental observations and notes be maintained in the field book, even though a data collector is used to record the observations.

b. Field Coordinate Geometry (COGO) computations. Most data collectors now have a full field capability to perform any surveying computation required. Some of the main field computational capabilities that are found on state-of-the-art data collectors include:

- Coordinate computations from radial direction-distance observations
- Multiple angle/direction adjustments
- Offset object correction (horizontal or vertical)
- EDM meteorological, slope, and sea level reductions
- Horizontal grid and datum transformations

- Vertical datum transformations
- GPS baseline reductions (static, kinematic)
- Traverse adjustments (various methods)
- Inverse and forward position computations
- Resections (2, 3 or more point adjustments)
- Level net adjustments (trig or differential)
- RTK site calibration adjustments (regression fits)
- Construction stake out (slope, horizontal & vertical curves, transition/spiral curves, etc.)

c. Feature coding and attributing. Data collectors are designed to encode observed topographic features with a systematic identification. Similar features will have the same descriptor code--e.g., "BS" for "backsight" and "EP" for "edge of pavement." Features that are recorded in the data collector can have additional attributes added. Attributes might include details about the feature being located (e.g., the number of lamps and height of a light pole).

d. Field graphic and symbology displays. Many field data collectors have symbology libraries which can be assigned to standard features, e.g., manholes, culverts, curb lines, etc. Plotted display of collected points with symbology can be viewed on the data collector display screen, or transferred to a portable laptop screen that has a larger viewing area. This allows for a visual view in the field of observed data in order to check for errors and omissions before departing the job site. This capability is, in effect, a modern day form of a plane table.

e. Data transfer. Digital survey data collected in the field is transferred from the data collector to a laptop or desktop PC for final processing and plotting in CADD (e.g., MicroStation, AutoCAD). Both original and processed data observations are transferred. Original (raw) data includes the unreduced slope distances, HIs, HRs, backsight and foresight directions, etc. Field processed data includes items such as reduced horizontal distances, adjusted coordinates, features, attributes, symbology, etc. Many field-finish software packages can generate level/layer assignments that will be compatible with CADD packages.

f. Reference. Additional details on data collectors and COGO are covered in Chapter 7 of this manual.

2-25. Methods of Delineating and Densifying Topographic Features

A variety of methods can be used to tie in planimetric features or measure ground elevations. Some type of systematic process is used to ensure full coverage of a job site--e.g., running cross-sections from a centerline baseline or a grid pattern. Feature accuracy will also vary: an invert elevation will be shot to 0.01 ft whereas ground shots on irregular terrain are recorded to the nearest 0.1 ft; the horizontal location of a building corner or road centerline will be to the nearest 0.01 ft but a tree can be positioned to the nearest foot.

a. Cross-section survey methods. Most site plan topographic surveys are performed relative to project baselines. This is often called the “right-angle offset technique” (Kavanagh 1997). A baseline is established along a planned or existing project axis (e.g., road centerline) using standard traverse control survey methods, as shown in Figure 2-9. Intermediate points are set and marked at regular intervals along the baseline (at 50-ft or 100-ft stations with intermediate stations added at critical points). The intermediate points are marked with 2x2 inch wooden hubs, PK nails, or temporary pins with flagging. Station hubs are occupied with a transit or total station and cross-sections are taken normal to the baseline alignment. Points along the cross-section offsets are shot for feature and/or elevation. Offset alignment is done either visually, with a right-angle glass, or transit, depending on the accuracy required. Distances along offsets are measured by chaining, stadia, or EDM (i.e., total station). Detailed notes and sketches of ground shots and planimetric features are recorded in a standard field book, electronic data collector, or both. Notekeeping formats will vary with the type of project and data being collected. General industry standard notekeeping formats should be used, such as those shown in any of the texts listed under Appendix A-2. Examples of selected topographic baseline notes are shown in Figures 2-10 and 2-11 and in Chapter 12.

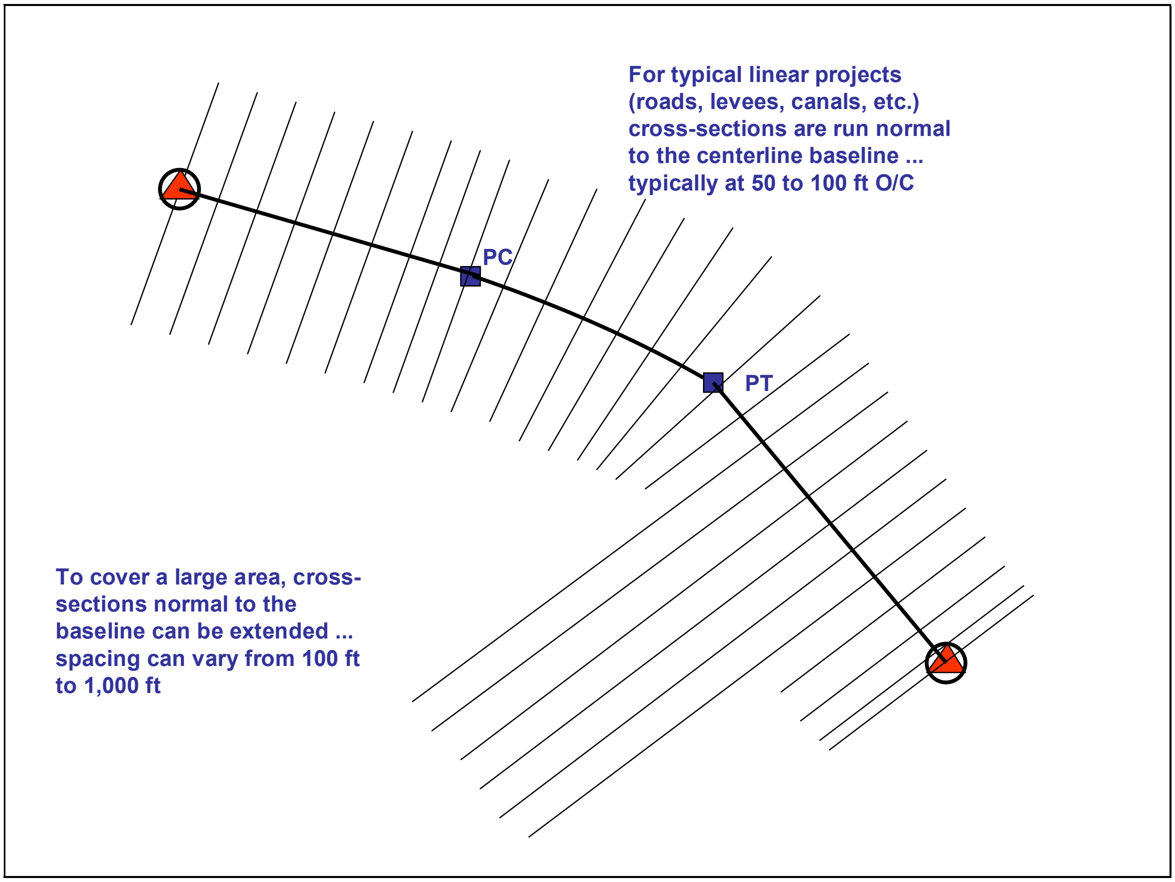


Figure 2-9. Illustration of cross-sections alignments run normal to established baselines

A grid pattern of cross-sections is also used for topographic survey of large areas, such as wetlands, orchards, swamps, etc. This is also illustrated in Figure 2-9 above where the cross-sections southeast of the PT extend a considerable distance from the baseline. In general, the maximum distance to extend the baseline is a function of the feature accuracy requirements and the precision of the survey instrument. For total stations, ground shots on a prism rod out to 1,000 ft and greater are usually acceptable. Transit stadia distances should not extend out beyond 500 ft. If coverage beyond 1,000 ft is needed, then additional baselines need to be run through the area and intermediate cross-sections should be connected between these baselines. (In current practice, this is rarely performed anymore--radial methods with a total station or RTK system are far more productive).

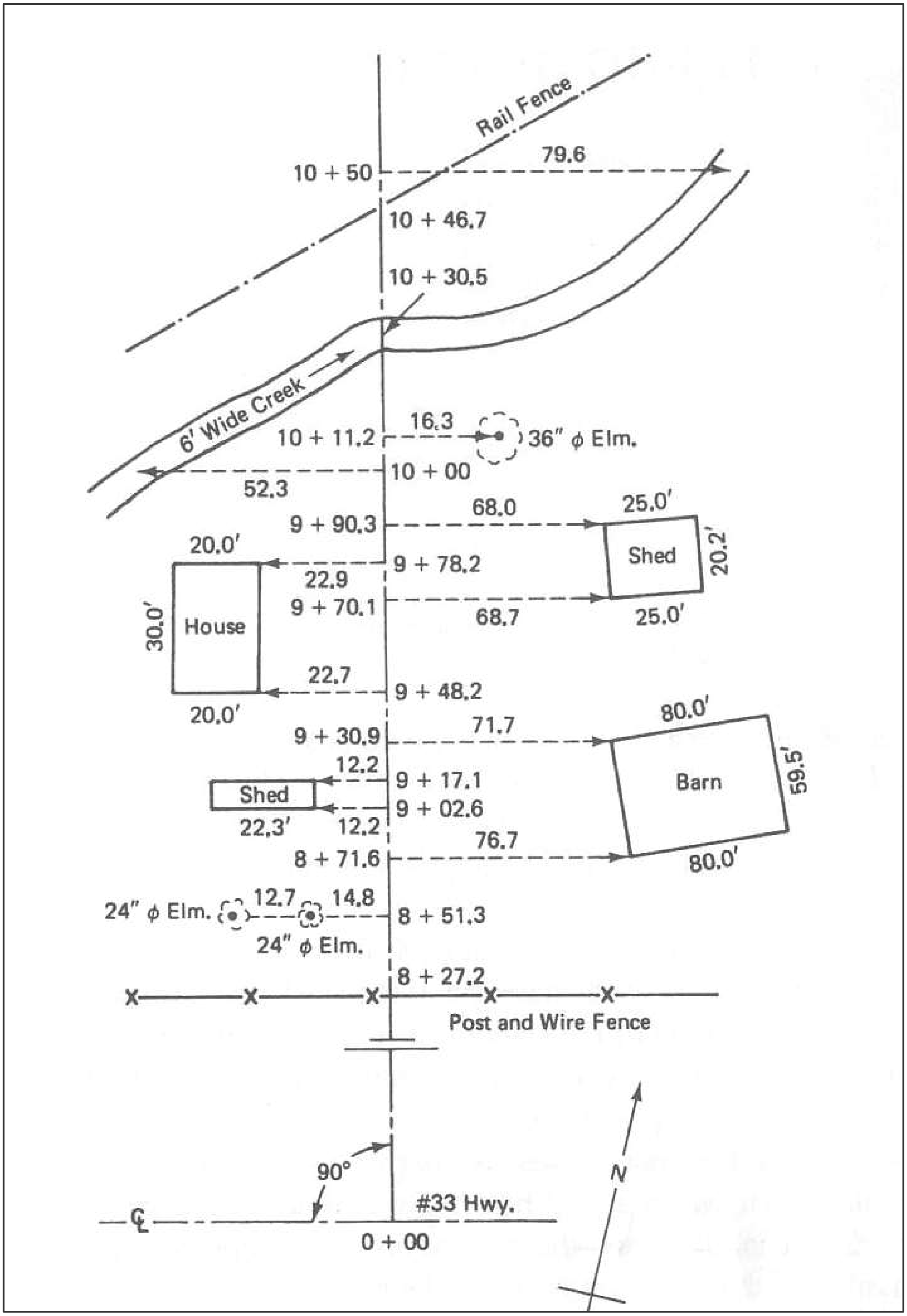


Figure 2-10. Sketch of profile line and cross-section (Kavanagh 1997)

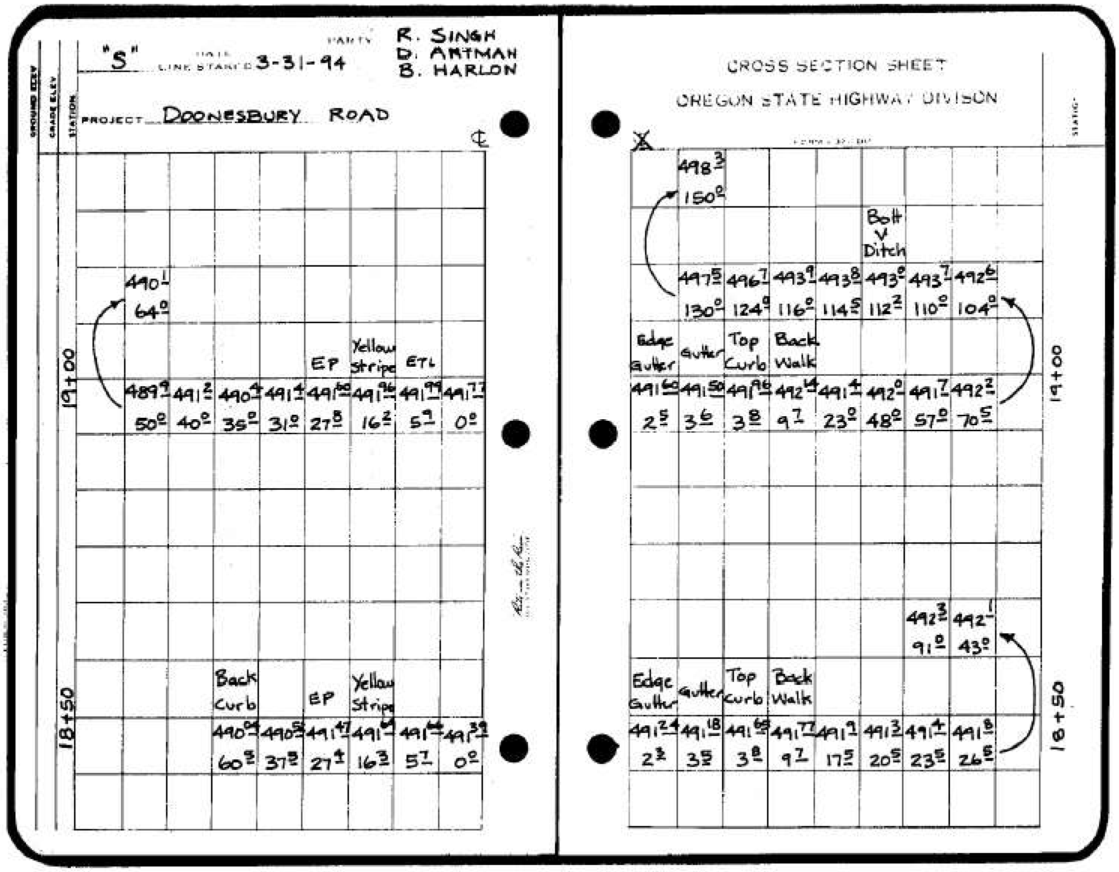


Figure 2-11. Example of field book notes showing location relative to centerline and elevation data from two cross-sections spaced 50-ft C/C (Oregon DOT 2000)

b. Radial survey methods. (Figure 2-12). Plane tables were especially suited to radial survey methods; thus, most surveys using total stations or RTK now utilize this technique. Radial observations are made with the instrument (total station or RTK base station) set up over a single point that has full project area visibility (or in the case of RTK, can encompass radio or cell phone ranges well beyond visible limitations with a total station). Thus, topographic features, baseline stakeout, and elevations can be surveyed without having to occupy separate stations along a fixed baseline. COGO packages will automatically compute radial distances and azimuths to linear or curved baseline stations, and visually guide the stakeout process. RTK surveys methods are a unique form of radial survey methods--RTK controller COGO packages are used to reduce GPS observations and guide alignment. Planimetric and ground elevation coverage is performed in a systematic pattern to ensure that the project site is adequately covered. This was straightforward on a plane table--the drawing could be viewed for omissions. On electronic data collector devices, verifying coverage before breaking down the instruments is not as easy. Data collector display screens are typically small and not all field data may have been collected using “field-finish” string (polyline) type coding.

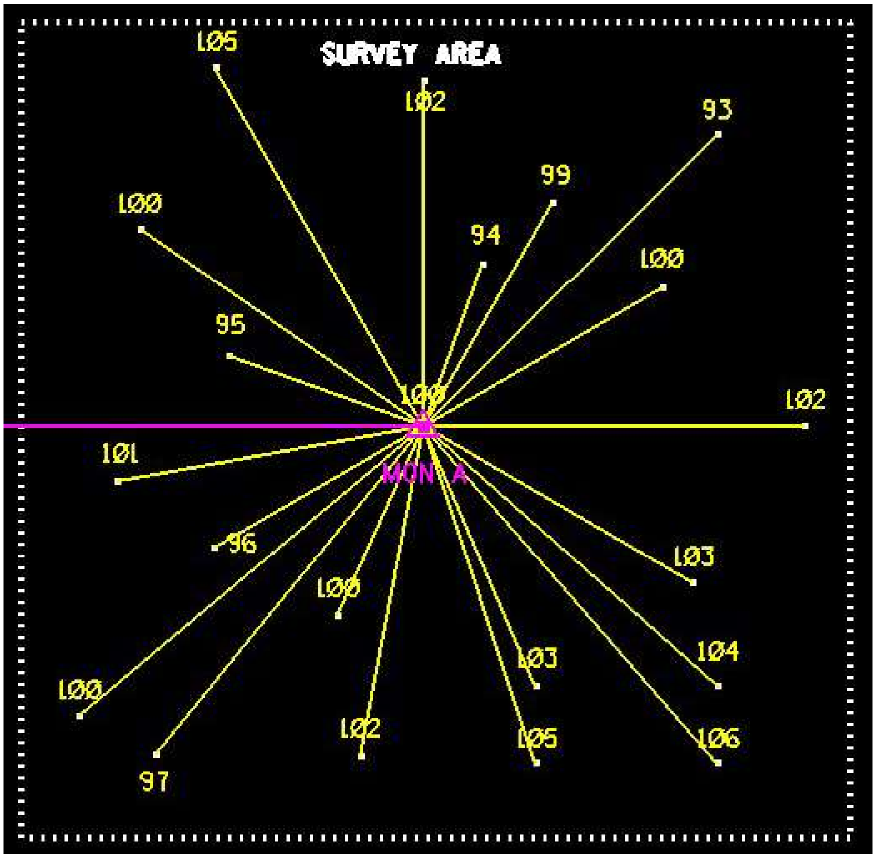


Figure 2-12. Topographic detail densification using radial survey methods--instrument set at point "MON A" and radial shot points (planimetric features or elevations) are observed

c. Planimetric features. Planimetric features are tied in using either cross-section or radial survey methods. The amount of detail required on a feature depends on the nature of the project and the size of the feature relative to the target scale. On small-scale topographic mapping projects, a generic symbol may be used to represent a feature; however, on a detailed drawing for this same project, the feature may be fully dimensioned. An example would be a 3 ft x 5 ft catch basin: on a 1inch = 400 ft scale map, this basin would be represented by a symbol at its center point but might be surveyed in detail (all four corner points located) on a 1inch = 30 ft site plan.

d. Topographic elevations and contours. A variety of survey methods are used to develop the terrain model for a given project area. The technique employed is a function of the type of survey equipment, the detail required, and specified elevation accuracy. In addition, the technique may depend on whether traditional contours or a digital terrain model (DTM) is required.

e. Contours from cross-sections. Contours can be directly surveyed on the ground or derived from a terrain model of spot elevations. When cross-section methods are employed, even contour intercepts along the offsets can be set in the field using a level rod. Alternatively, elevations can be taken at intervals along the cross-section where changes in grade or breaklines occur, and contour intercepts interpolated over the linear portions. If abrupt changes in grade (or breaks in grade) occur between cross-section stations, then supplemental cross-sections may be needed to better represent the terrain and provide more accurate cut/fill quantity takeoffs.

f. Contours from radial surveys--spot elevation matrices. It is often more efficient to generate contours from a DTM based on spot elevations taken over a project area. These surveys are normally done with a total station or RTK system; however, older transit-stadia or plane table methods will also provide the same result. The density of spot elevations is based on the desired contour interval and terrain gradient. In some instances, an evenly spaced grid of spot elevations may be specified (so-called "post" spacing). Flat areas require fewer spots to delineate the feature. Breaklines in the terrain are separately surveyed to ensure the final terrain model is correctly represented. Data points can be connected using triangular irregular network (TIN) methods and contours generated directly from the TIN in various CADD packages (MicroStation InRoads, AutoCAD, etc.). The generated DTM or TIN also provides a capability to perform "surface-to-surface" volume computations.

g. DTM generation from breakline survey technique. The following guidance is excerpted from the California Department of Transportation (CALTRANS) *Surveys Manual*. It describes a technique used by CALTRANS to develop DTMs on total station topographic surveys.

A DTM is a representation of the surface of the earth using a triangulated irregular network (TIN). The TIN models the surface with a series of triangular planes. Each of the vertices of an individual triangle is a coordinated (x,y,z) topographic data point. The triangles are formed from the data points by a computer program which creates a seamless, triangulated surface without gaps or overlaps between triangles. Triangles are created so that their sides do not cross breaklines. Triangles on either side of breaklines have common sides along the breakline.

Breaklines define the points where slopes change in grade (the intersection of two planes). Examples of breaklines are the crown of pavement, edge of pavement, edge of shoulder, flow line, top of curb, back of sidewalk, toe of slope, top of cut, and top of bank. Breaklines within existing highway rights of way are clearly defined, while breaklines on natural ground are more difficult to determine. DTMs are created by locating topographic data points that define breaklines and random spot elevation points. The data points are collected at random intervals along longitudinal break lines with observations spaced sufficiently close together to accurately define the profile of the breakline. Like contours, break lines do not cross themselves or other break lines. Cross-sections can be generated from the finished DTM for any given alignments.

Method: When creating field-generated DTMs, data points are gathered along DTM breaklines, and randomly at spot elevation points, using the total station radial survey method. This method is called a DTM breakline survey. Because the photogrammetric method in most cases is more cost effective, gathering data for DTMs using field methods should be limited to small areas or to provide supplemental information for photogrammetrically determined DTMs. The number of breaklines actually surveyed can be reduced for objects of a constant shape such as curbs. To do this, a standard cross section for such objects is sketched and made part of the field notes. Field-collected breaklines are identified by line numbers and type on the sketch along with distances and changes in elevation between the breaklines. With this information in the field notes, only selected breaklines need to be located in the field, while others are generated in the office based on the standard cross section. Advantages of DTM breakline surveys:

- *Safety of field crews is increased because need to continually cross traffic is eliminated.*
- *Observations at specific intervals (stations) are not required.*
- *New sets of cross sections can be easily created for each alignment change.*

DTM survey guidelines:

- *Remember to visualize the TIN that will be created to model the ground surface and how breaklines control placement of triangles.*
- *Use proper topo codes, point numbering, and line numbers.*
- *Use a special terrain code (e.g., 701) for critical points between breaklines, around drop inlets and culverts, and on natural ground in relatively level areas.*
- *Make a sketch of the area to be surveyed identifying breaklines by number.*
- *Do not change breakline codes without creating a new line.*
- *Take shots on breaklines at approximately 20 m intervals and at changes in grade.*
- *Locate data points at high points and low points and on a grid of approximately 20 m centers when the terrain cannot be defined by breaklines.*
- *If ground around trees is uniform, tree locations may be used as DTM data points by using a terrain code of 701.*
- *Keep site distances to a length that will ensure that data point elevations meet desired accuracies.*
- *Gather one extra line of terrain points 5 to 10 m outside the work limits.*

Accuracy Standard: Data points located on paved surfaces or any engineering works should be located within ±10 mm horizontally and ± 7 mm vertically. Data points on original ground should be located within ± 30 mm horizontally and vertically.

Checking: Check data points by various means including reviewing the resultant DTM, reviewing breaklines in profile, and locating some data points from more than one setup.

Products: The Surveys Branch is responsible for developing and delivering final, checked engineering survey products, including DTMs, to the survey requestors. Products can be tailored to the needs of the requestor whenever feasible, but normally should be kept in digital form and include the following items:

- *Converted and adjusted existing record alignments, as requested. (CAiCE project subdirectory)*
- *Surveyed digital alignments of existing roadways and similar facilities. (CAiCE project subdirectory)*
- *CAiCE DTM surface files. (CAiCE project subdirectory)*
- *2-D CADD MicroStation design files, .dgn format.*
- *Hard copy topographic map with border, title block, labeled contours, and planimetry.*
- *File of all surveyed points with coordinates and descriptions. (CTMED, .rpt, format)*

h. Utility survey detail methods. It is important to locate all significant utility facilities. Utilities are surveyed using either total station or RTK techniques. The CALTRANS *Surveys Manual* recommends that accuracy specifications for utilities that are data points located on paved surfaces or any engineering works should be located within ±10 mm horizontally and ±7 mm vertically. Data points on original ground should be located within ±30 mm horizontally and vertically. The following are lists of facilities and critical points to be located for various utilities--as recommended in the CALTRANS *Surveys Manual*.

Oil and Gas Pipelines

- *Intersection point with centerlines and/or right of way lines*

- *For lines parallel to right of way – location ties necessary to show relationship to the right of way lines*
- *Vents*
- *Angle points*
- *Meter vaults, valve pits, etc.*

Water and Sewer Lines
- *Intersection point with centerlines and/or right of way lines*
- *For lines parallel to right of way – location ties necessary to show relationship to the right of way lines*
- *Manholes, valve boxes, meter pits, crosses, tees, bends, etc.*
- *Elevation on waterlines, sewer inverts, and manhole rings*
- *Fire hydrants*
- *Curb stops*
- *Overhead Lines*
- *Supporting structures on each side of roadway with elevation of neutral or lowest conductor at each centerline crossing point.*
- *On lines parallel to roadway, supporting structures that may require relocation, including overhead guys, stubs, and anchors*

Underground Lines
- *Cables/lines (denote direct burial or conduit, if known), etc.*
- *Manholes, pull boxes, and transformer pads*
- *Crossing at centerline or right of way lines*
- *For lines parallel to right of way – location ties as necessary to show relationship to the right of way lines*

Railroads
- *Profile and location 60 m each side of the proposed roadway right of way lines*
- *Switch points, signal, railroad facilities, communication line locations, etc.*

Checking: Utility data should be checked by the following means:
- *Compare field collected data with existing utility maps*
- *Compare field collected data with the project topo map/DTM*
- *Review profiles of field collected data*
- *Include field collected data, which have elevations, in project DTM*
- *Locate some data points from more than one setup*

i. Archaeological Site/Environmentally Sensitive Area Surveys (CALTRANS). Archaeological and environmental site surveys are performed for planning and engineering studies. Surveys staff must work closely with the appropriate specialists and the survey requestor to correctly identify archeological and environmentally sensitive data points.

Method: Total station radial survey, GPS fast-static, kinematic, or RTK.

Accuracy Standard: Data points located on paved surfaces or engineering works should be located within ±10 mm horizontally and ±7 mm vertically. Data points on original grounds should be located within ±30 mm horizontally and vertically. Review field survey package for possible higher required accuracy.

Checking: Check data points by various means including, reviewing the resultant DTM, reviewing breaklines in profile, and locating some data points from more the one setup.

Products:
- *3-D digital graphic file of mapped area*
- *Hard copy topographic map with border, title block, and planimetry (contours and elevations only if specifically requested)*
- *File of all surveyed points with coordinates and descriptions*

j. Spot Location or Monitoring Surveys (CALTRANS). Monitoring surveys are undertaken for monitoring wells, bore hole sites, and other needs.

Method: Total station radial survey, GPS fast static or kinematic

Accuracy Standard: Data points located on paved surfaces or any engineering works should be located within ±10 mm horizontally and ±7 mm vertically. Data points on original ground should be located within ±30 mm horizontally and vertically.

Checking: Observe data points with multiple ties.

Products:
- *File of all surveyed points with coordinates and descriptions*
- *Sketch or map showing locations of data points*

k. Vertical Clearance Surveys (CALTRANS). Vertical clearance surveys are undertaken to measure vertical clearances for signs, overhead wires, and bridges.

Method: Total station radial method.

Accuracy Standard: Data points located on paved surfaces or any engineering works should be located within ±10 mm horizontally and ±7 mm vertically. Data points on original ground should be located within ±30 mm horizontally and vertically.

Checking: Observe data points with multiple ties.

Products:
- *File of all surveyed points with coordinates and descriptions*
- *Sketch or map showing vertical clearances*

2-26. General QC and QA Guidance on Topographic Data Collection and Drawings

The following guidance is excerpted from the Woolpert, Inc. "Survey Manual." It contains a "checklist" of critical features and attributes that must be collected for various types of utilities, roads, boundaries, facilities, HTRW sites, and other structures. It is followed by a "Drawing Annotation Checklist" that provides general guidance on attributing various features.

INTRODUCTION: Topographic surveys are the basis for the engineering, planning, and development plans. It is critical that the information shown on the plans be correct and complete. It is also very important to understand the intended use or accuracy requirements needed by the user of the plan along with the size of the project. This information can be useful to determine whether the project should be collected by aerial mapping of ground run surveys. If the plan is to be used for engineering design then the field survey will likely include pavement sections and utility locations.

ITEMS INCLUDED ON SURVEYS

Topography (General)
- *Performed by using trigonometric techniques with the Total Station or digitized by aerial photography.*
- *Provide and identify the natural relief of the ground, and man-made structures.*
- *Topography (Location)*
- *Natural*
- *Establish the location of "top of bank", "toe of slope," and centerline of all streams or creeks.*
- *Provide cross sections at specified intervals – typically 20 meters or 50 ft.*
- *Provide a "spot grade" shot +/- 30 ft away from "top of bank" at the cross section interval*
- *Provide a "top of water" shot at every 1000 ft interval – record date and time if tidal area or recent weather events if not tidal.*
- *Sizes of trees will be identified by common name and/or scientific name, and their diameter will be measured at DBH or "diameter at breast height."*
- *Provide location of isolated or cultivated trees*
- *Provide location of edge of woods at outside "drip line."*
- *Locate all high points and low points along ridges and valleys.*
- *Note: Some circumstances may require the location of: spoil piles, sink holes, standing water, caves, and unusual rock outcrops.*
- *Note: Some circumstances may require the locations of the "thread" or "thalweg line" when providing a profile of a stream.*
- *Wetlands*
- *Delineation of a wetland can be located only after flags have been set by an environmental scientist from either the Army Corps of Engineers or Department of Transportation.*

Ditches and Drainage Features
- *Establish the location of "top of bank", "toe of slope," and centerline of all ditches.*
- *Provide cross sections at specified intervals – typically 20 meters or 50 ft.*
- *Provide a "spot grade" shot – typically +/- 30 feet away from "top of bank" at the cross section interval.*

- *Locate any concrete or asphalt: flumes, V-ditches, UD – drains or channels.*
- *Locate all yard drop inlets and curb drop inlets.*
- *Locate all headwalls and wing walls.*
- *Measure the diameter and note the type of all pipes.*
- *Provide location and elevation on invert (flow line) of pipe.*

Storm and Gravity Sanitary Sewers
- *Obtain elevations and location on the tops of manholes or drop inlets.*
- *Measure readings (downs) from rim of manhole to inverts*
- *Locate and provide elevations on inverts and manholes on the next structure out of the limits.*
- *Obtain location and elevations on inverts on box culverts*
- *Obtain location and elevations on inverts on ends of flared-end-section pipes*
- *Locate sanitary sewer clean outs*
- *Locate and describe sanitary sewer pump stations (lift stations).*
- *Locate approximate areas of septic fields and tanks.*

Roads
- *Locate and measure all curb and gutter features: Back of curb, flow line, and edge of gutter pan.*
- *Note size and type of curb and gutter.*
- *Provide location of edge of pavement at specified intervals – typically 20 meters or 50 ft.*
- *Note size and type of pavement.*
- *Provide location of centerline or "crown" of road.*
- *Obtain and locate all entrances.*
- *If concrete pavement has been overlaid with asphalt, measure approximate depth of overlay.*
- *Locate and note types of guardrails.*
- *Locate and provide elevations at the base of Jersey barrier.*

Railroads
- *Provide location of tracks with elevations at specified intervals--typically 20 meters or 50 ft in a curve. Note: Some special circumstances may also include location and elevations for the ballast rock and railroad bed.*
- *Obtain location of all switches.*
- *Obtain location of all mileposts. Note: Most crossing signals provide distances to closest milepost. If a railroad milepost cannot be located, the closest railroad spur must be located and tied.*
- *Obtain location of all signal equipment.*
- *Obtain location of all Right-of-way monuments.*
- *Obtain location, size, and type of culverts under the railroad.*
- *Secure a copy of the railroad right-of-way map.*

Fences
- *Provide location, type, and height of fence.*
- *Common types of fences are split rail, wood privacy, chain link, woven wire, barbed wire, etc.*

Cemeteries
- *Location of cemetery boundary must be shown.*
- *Locate graves coincident with the Right-of-way and survey centerline.*
- *Provide an approximate count of the number of graves.*

Automobile Graveyards
- *Locate outside limits and note approximate number of automobiles.*

Signs
- *Locate and describe all overhead truss signs.*
- *Locate and describe all overhead cantilever signs.*
- *Locate and describe all breakaway I-beam traffic signs.*
- *Locate and describe all traffic signals.*
- *Locate and describe all historical markers – recording identity numbers.*
- *Locate, measure, and describe in detail all advertising signs or commercial billboards. It is imperative to note the owner and the license number.*

House & Building Location
- *Locate all dwellings and buildings at the wall or footer line and note/dimension the overhang.*
- *Describe as dwellings, buildings, restaurants, etc.*
- *Identify structure address: example) house or box number.*
- *Describe the height of structures: example) one story, two story, or split-level.*
- *Describe the type of construction: example) brick, wood frame.*
- *Locate and describe all porches, decks, carports, utility buildings, and driveways.*

Utility Items--Above Ground Utility Location
- *Utility poles and guy wire anchors – recording number and owner.*
- *Light poles – recording number and owner.*
- *Cable TV pedestals – recording number and owner.*
- *Electric – cabinet, transformer, junction box, hand hole, witness post, meter, transmission tower, and Sub-Stations (note: do not enter facility).*
- *Water meters, valves, vaults, manholes, blow off valves, fire hydrants and witness posts.*
- *Gas meters, valves, test stations, and witness posts.*
- *Force main air vents and witness posts along line as well as valves and emergency pump connections at pump station facility (note: do not enter facility).*
- *Steam pipes and steam manholes.*
- *Petroleum pipes, witness posts, and pumping stations (note: do not enter facility).*
- *Communication or telephone manholes, pedestals, hand holes, and witness posts – recording number and owner.*
- *Traffic control signals, manholes, cabinets, junction boxes, and hand holes.*

Political Boundaries & Road Names
- *Provide location of all monuments of city or town corporate limits.*
- *Obtain the location of all monuments pertaining to county or state lines.*
- *Locate all street name signs and route number identifiers.*

Government Survey Control

- *Locate all government benchmarks.*
- *Locate all government triangulation, trilateration, and traverse stations.*
- *Locate all government reference marks and azimuth marks.*
- *Locate all state Right-of-way monuments.*

Property Data – (If required)

- *Obtain Right-of-way plans from State Location and Design Engineer.*
- *Obtain pertinent data from court records such as; subdivision plats, parcel, or tract deeds and plats, and tax assessor's cards and maps.*
- *Provide location of all property monuments called for in the deed as needed per scope*
- *Provide location of all easements.*

Hazardous Material/Waste Sites

- *Typically, all hazardous waste sites or potential waste sites will be noted.*
- *Obtain site plan of suspected area*
- *Note and record pertinent information on location of underground storage tanks, filler caps, monitoring wells and caps.*

Set TBMs

- *Obtain and verify vertical datum as per scope: Assumed, City datum, NGVD29, or NAVD88.*
- *A minimum of two temporary benchmarks will be set on private topographic surveys.*
- *TBMs will be set an interval of 1000 ft. to 1500 ft. on typical corridor surveys.*

<u>*DRAWING ANNOTATION CHECKLIST*</u>

- *Advertising Signs (Billboards): Locate if needed and show license number and owner (small license plate).*

- *Automobile Graveyards: Locate the outside limits and annotate.*

- *Brush, shrubbery, woods: Annotate as dense, light, mixed, etc., and type. Example: (Tree types). Description of trees: describe the type of tree, not just hardwood and pine unless it cannot be identified. Use "Shrub" instead of "Bush" in all cases.*

- *Buildings: Locate at the overhangs and annotate type brick, frame, etc., the height (one story, two story, etc.), and name if commercial. Carports, porches, steps, walks, etc., will also be shown. Example: (1 Story frame dwelling #3098), (2 Story brick building #4139); Building numbers need to be shown. If no number is visible note that, do not leave it blank. Sheds are structures with a roof, and four support posts; Buildings are structures enclosed by four sides, and a door; a Dwelling is a structure that someone lives in; a Commercial Building is a business; and a Restaurant is a structure that someone eats in. The occupant of a Commercial building shall also be identified, i.e. Exxon or First Union Bank. Strip malls will be called out as such or by the name of the shopping center.*

- *Bridges: Annotate type, with deck.*

- *Curbs and Gutters: Annotate type and size.*

- *Cemeteries: Locate the extremities, the closest grave to the centerline and annotate the approximate number of graves.*

- *Concrete or Paved Ditches: Annotate type and width. Flow elevations and directions will be secured by a field survey.*

- *Concrete or Paved Flumes: Annotate type and width. Flow elevations and directions will be secured by a field survey.*

- *Curbs: Annotate type and size.*

- *Culverts: Annotate type, size, secure invert elevations, and direction of flow.*

- *Dams: Annotate type.*

- *Entrances: Annotate type (soil, gravel, asphalt, etc.).*

- *Electric Manholes and junction boxes: Annotate.*

- *Endwalls and Headwalls: Annotate type.*

- *Fences: Annotate Height and type (wood, wire, or chain link), no split rail or woven wire.*

- *Fire Hydrants: Annotate.*

- *Guardrails: Annotate type.*

- *Guy Wires: Need to be annotated and located (number and furthest wire if more than one).*

- *Government Benchmarks, Triangulation Stations, Traverse Stations, Azimuth Marks, and Reference Marks: Annotate.*

- *Historical Marks: Annotate identification numbers.*

- *High Voltage Transmission Lines: Annotate. Electric transmission lines should be shown on the survey. Show one Tower outside the limits. List the number of lines on each tower. We do not need to show the location of the overhead lines.*

- *Light poles: Should be described differently, based upon use. The light poles along roadways are to be called out as "Street Light"; the lights in a shopping center, at a service station, around a hotel are to be shown as "Security Light." The light poles in someone's yard would be shown as "Lamp Post" and ground lights illuminating signs, etc. would be shown as "Outdoor Lights."*

- *Mile Markers: Annotate*

- *Names of all cities, towns, and villages must be annotated and all corporate limits, county and state lines located and annotated.*

- *Outlet Ditches: Annotate with directions of flow.*

- *Pavements: Annotate type and if concrete covered with asphalt, make notations.*

- *Pipes: Annotate type, size, invert elevations, and direction flow.*

- *Property Data: Corners will be located and annotated. All pins within the limits of the survey should be obtained if possible, especially each lot within a subdivision, not just pc and pt points on the subdivisions right-of-way.*

- *Ponds and Lakes: Annotate and collect DTM data inside the edge of water line styles.*

- *Roads: Annotate route numbers and street names and type.*

- *Right-of Way Monuments: Annotate.*

- *Railroads: Annotate owners, right-of-way, and distance to the nearest milepost.*

- *Sewage Disposal and Water Supply: Annotate for each individual developed property, privy, well, sewer clean outs, water meters, drain fields, septic tanks, etc. See homeowner if necessary.*

- *Special Signs: Annotate overhead truss, signal traffic lights, railroad protective devices, etc. (No street signs or speed limit signs are needed). Location and description of all other signs is required. Private signs should be picked up and described, as well as the type of supports, concrete pads or bases, and heights (especially the tall service station and restaurant signs that can be seen from the interstates). All green signs along interstate should be shown and described as on wood posts or steel breakaway posts, and if they are on concrete or not. Street signs should be picked up and identified as "Street Sign." Reflector posts, curve signs, speed limit signs, other delineator signs should not be picked up. If they show up in the mapping put an X through them.*

- *Storm and Sanitary Sewers: Annotate type. Example: SMH= Sanitary, SSMH=Storm, DI, etc. Secure rim elevations, inverts and/or flow lines of all structures. For curb drop inlets show elevation at low point of the throat, usually the center of actual box and measure the length of the throat.*

- *Telephone M.H.'s, pedestals, handholes: Annotate.*

- *Trees: Annotate type and size with the diameter measured three feet above the ground. If unsure of type, hardwood or pine will do.*

- *Utility Poles and Pedestals: Annotate number and owner initials. Include information if pole has light or transformer. Example: T-Ped-#R-1680, B.A. (Bell Atlantic) PP-AB-53, V.P. (Virginia Power).*

- *Walls: Annotate type, height, and width.*

- *Witness posts: annotate type.*

- *Identify gas station filler caps, monitoring wells and locate concrete pads around them.*

- *Identifying areas of possible hazardous materials and type of possible contamination.*

- *Use common sense…annotate, edit and/or revise areas not covered in the above and correct all discrepancies in mapping.*

- *Set TBM's or BM's approximately 1500 feet apart. They can be on the centerline. They should also be at all drainage crossings (canals, etc.) and bridges.*

- *Use discretion when setting nails in trees for references. Do not use ID caps on private property.*

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.

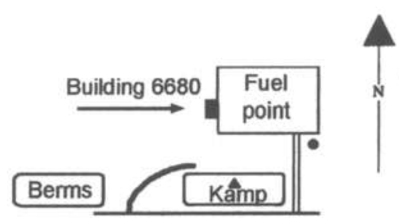
The station is a Type 70 monument protruding 20 cm above the ground and is located atop a burm.

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** JUL 2001 **1959** 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′	
9/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.

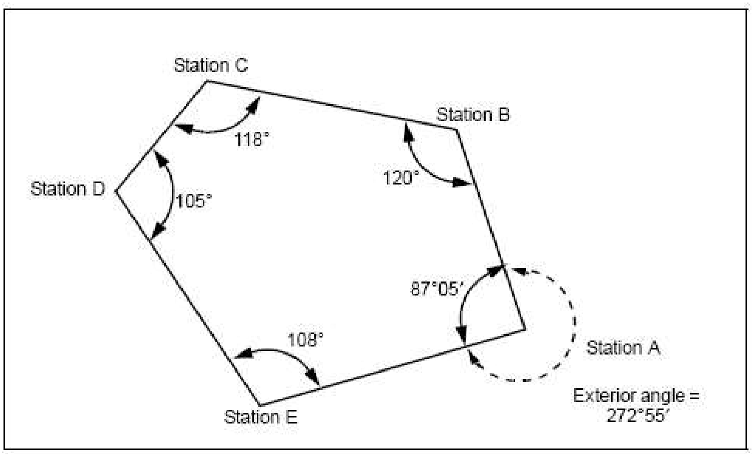
3-4. Bearing and Azimuth Determination

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.



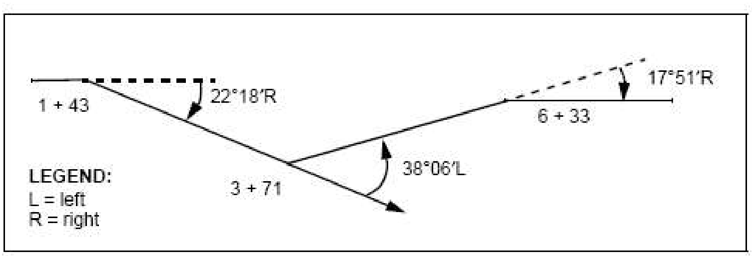


Figure 3-4. Interior angles on a closed traverse (top) and deflection angles on an open traverse (bottom) (FM 3-34.331)

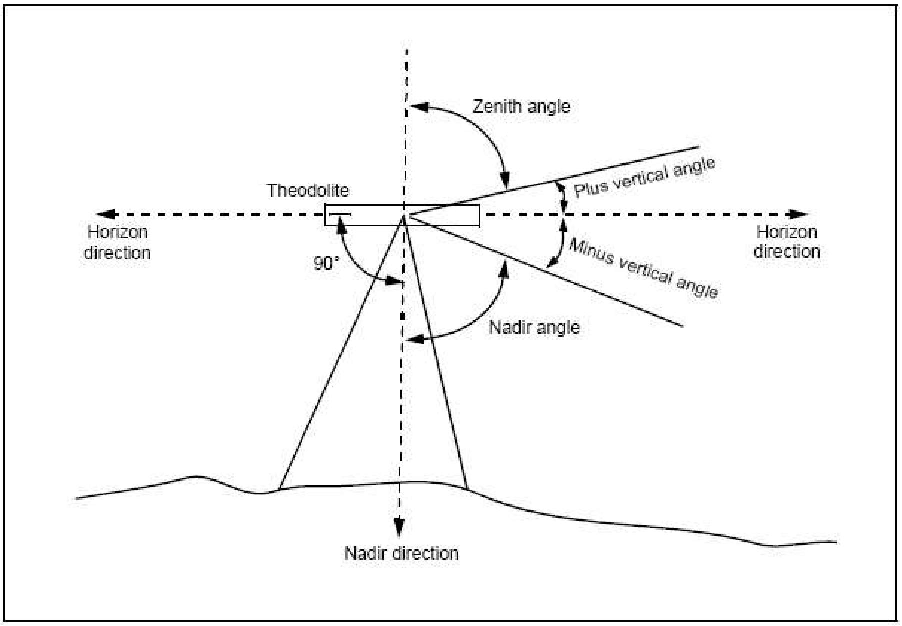


Figure 3-5. Reference directions for vertical angles--Horizontal, Zenith, and Nadir (FM 3-34.331)

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.

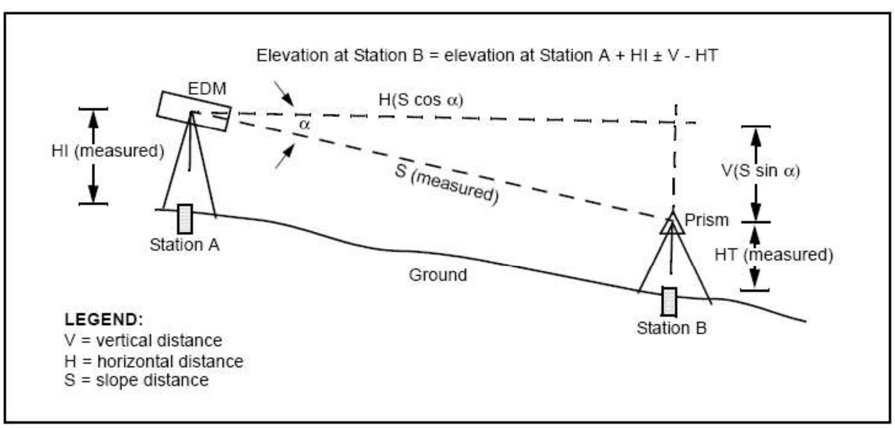


Figure 3-6. Geometry of an EDM measurement (FM 3-34.331)

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.

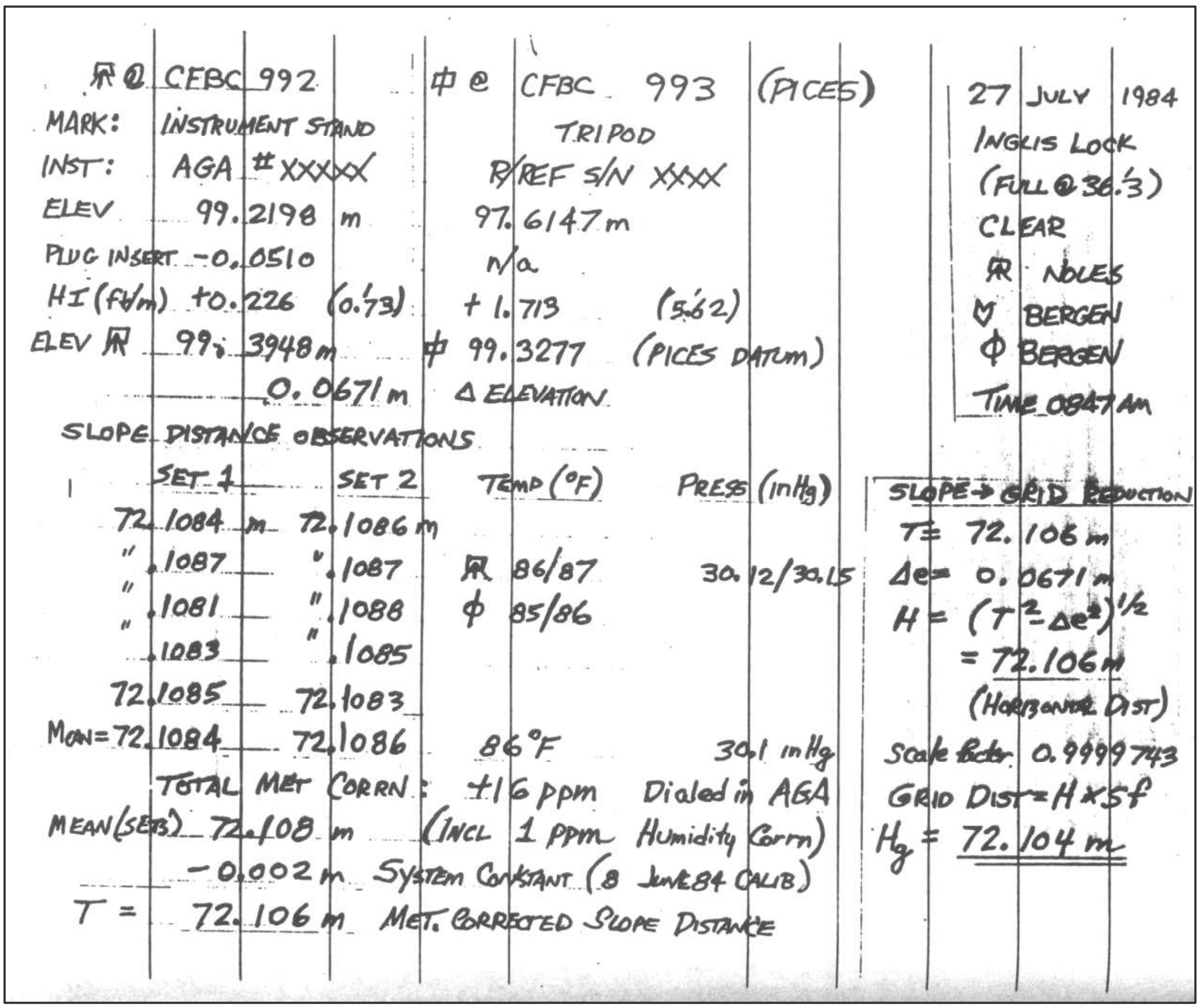


Figure 3-7. Horizontal distance observations and reductions--manual computations in field book (Jacksonville District)

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:

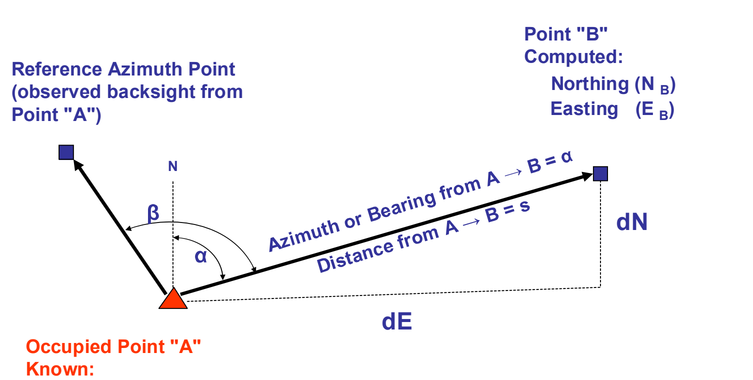
- The use of a prism typically provides an indicated distance longer than the true value. Applying a negative correction will compensate for this effect. Each prism should have its own constant or correction determined individually, and a master file should be maintained.

- 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.



$$N_B = N_A + s \cdot \cos\alpha$$
$$E_B = E_A + s \cdot \sin\alpha$$

Northing (N_B) = Northing (N_A) + s · cos α
Easting (E_B) = Easting (E_A) + s · sin α

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:

$$dN = s \cdot \cos(\alpha)$$
$$dE = s \cdot \sin(\alpha)$$

(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.

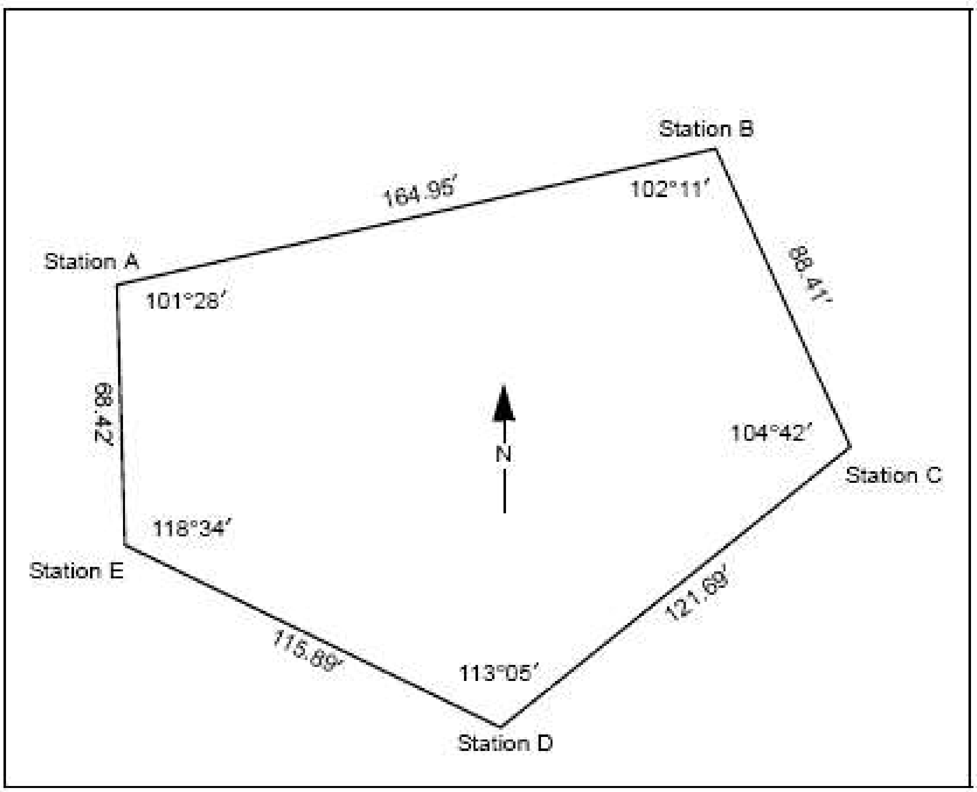


Figure 3-9. Closed Traverse--Looped (Station A fixed) or Connecting (Stations A and B fixed)

(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.

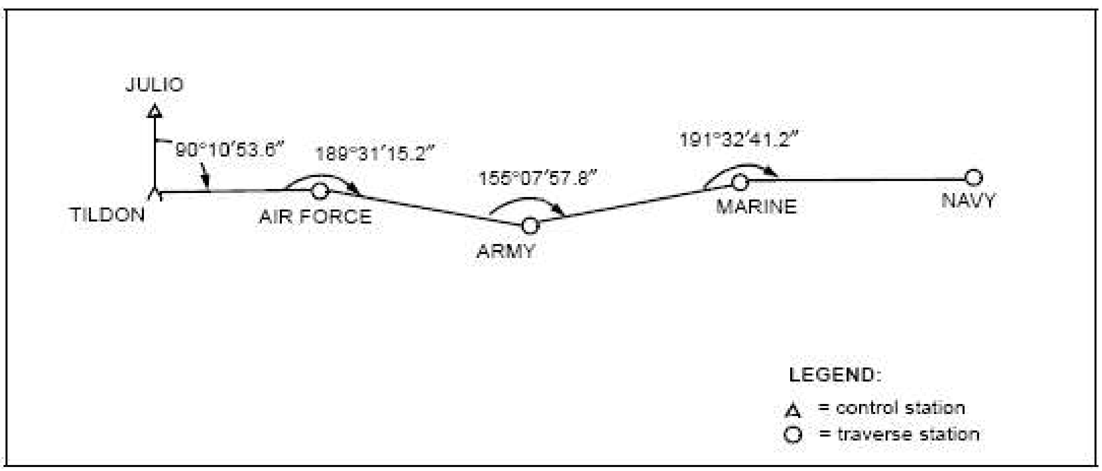


Figure 3-10. Open traverse

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.

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

where
l^ = 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:

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

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.

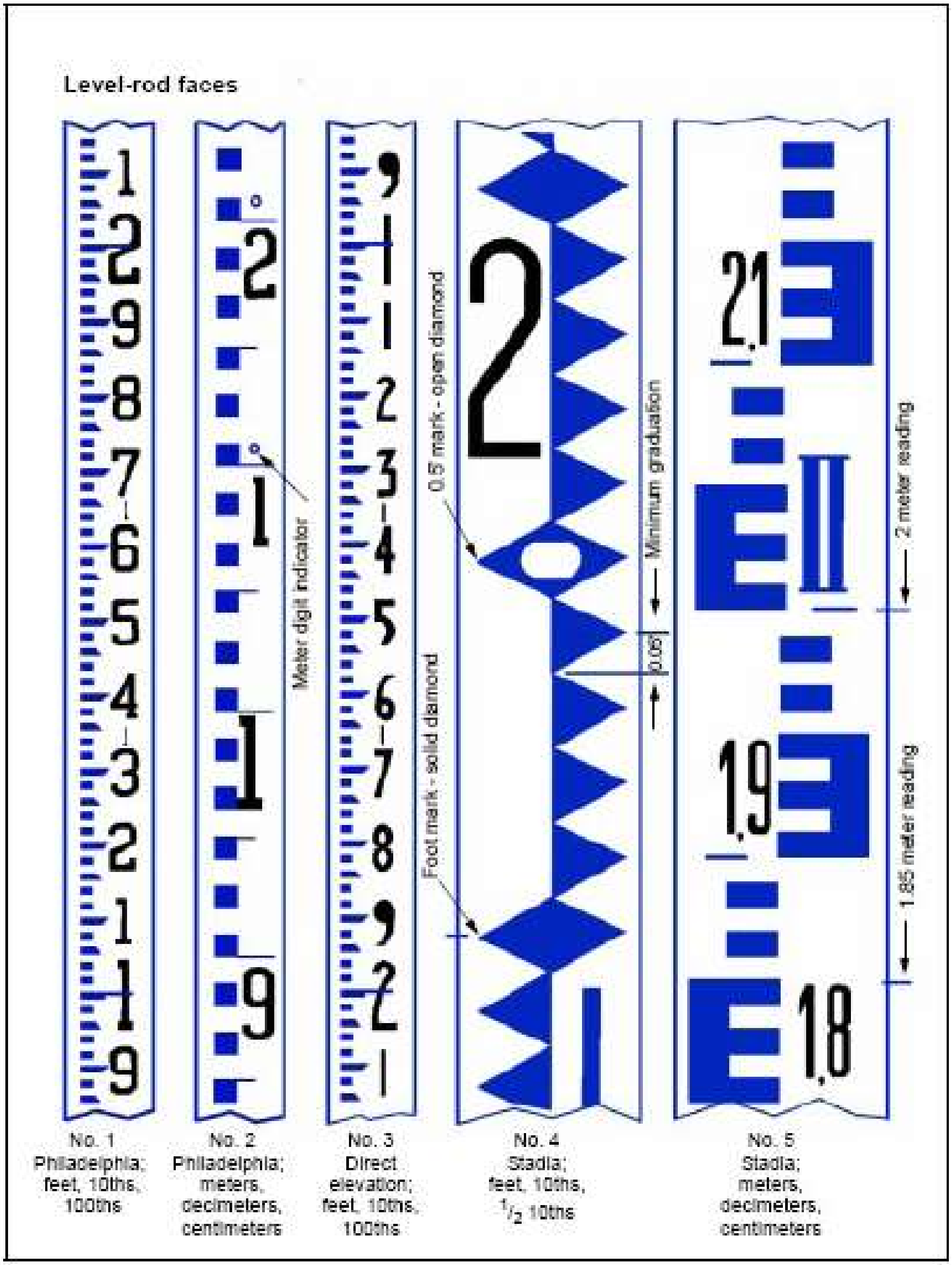


Figure 3-13. Traditional rectangular cross-section leveling rods showing a variety of graduation markings (FM 3-34.331)

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.

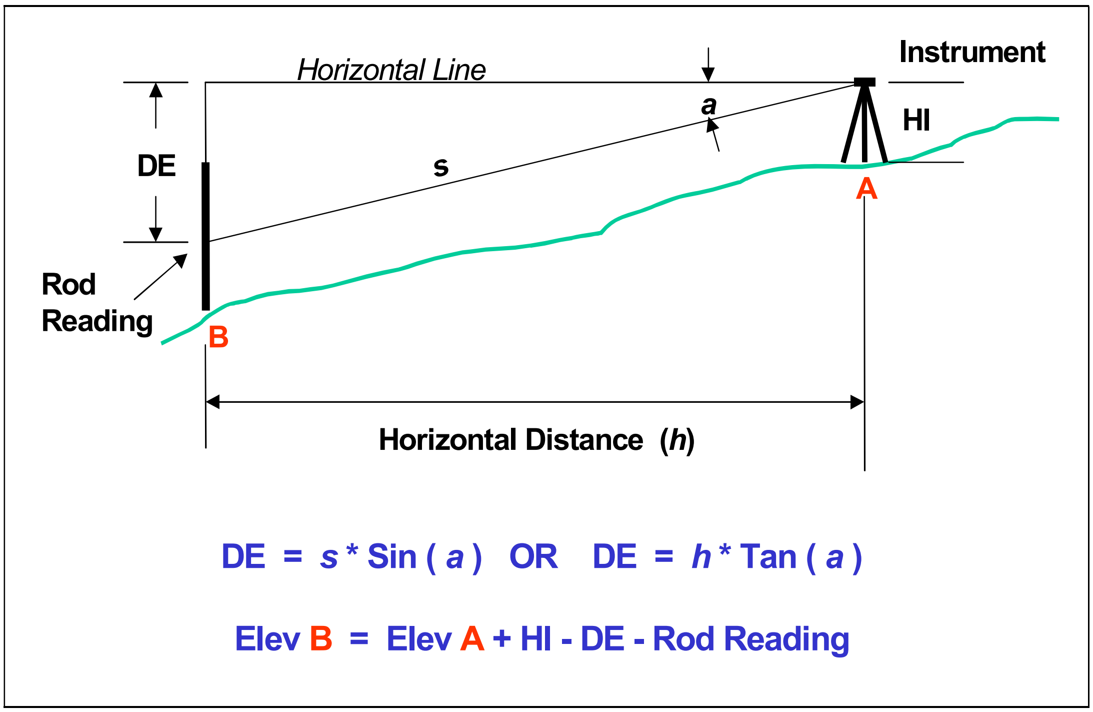


Figure 3-14. Trigonometric leveling

(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.

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

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.

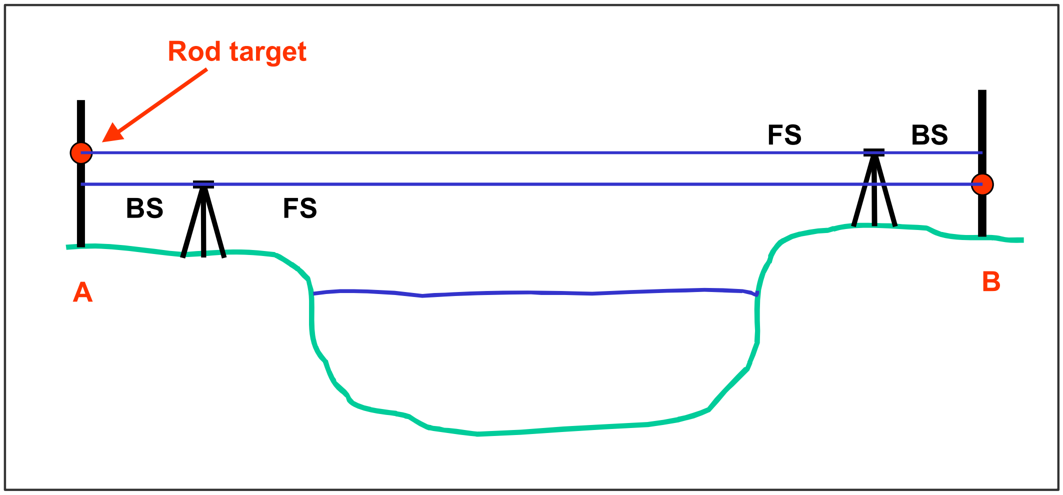


Figure 3-15. Reciprocal leveling for river crossing

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						

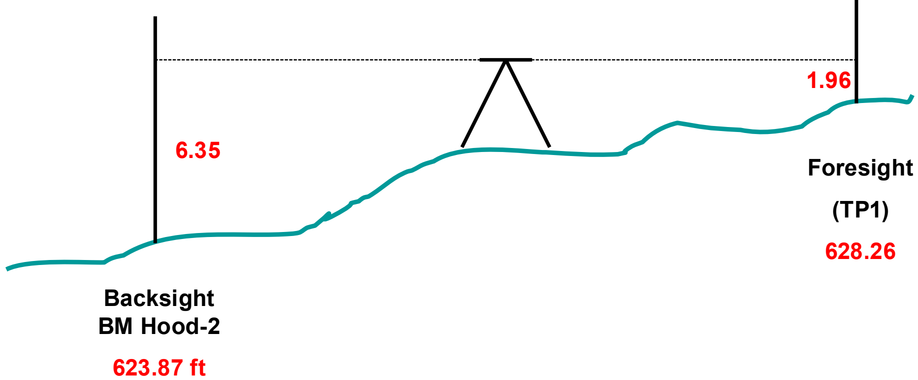


Figure 3-16. Differential leveling--example of one setup between benchmark and TP 1 (Standard field book recording format shown at top of sketch)

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.

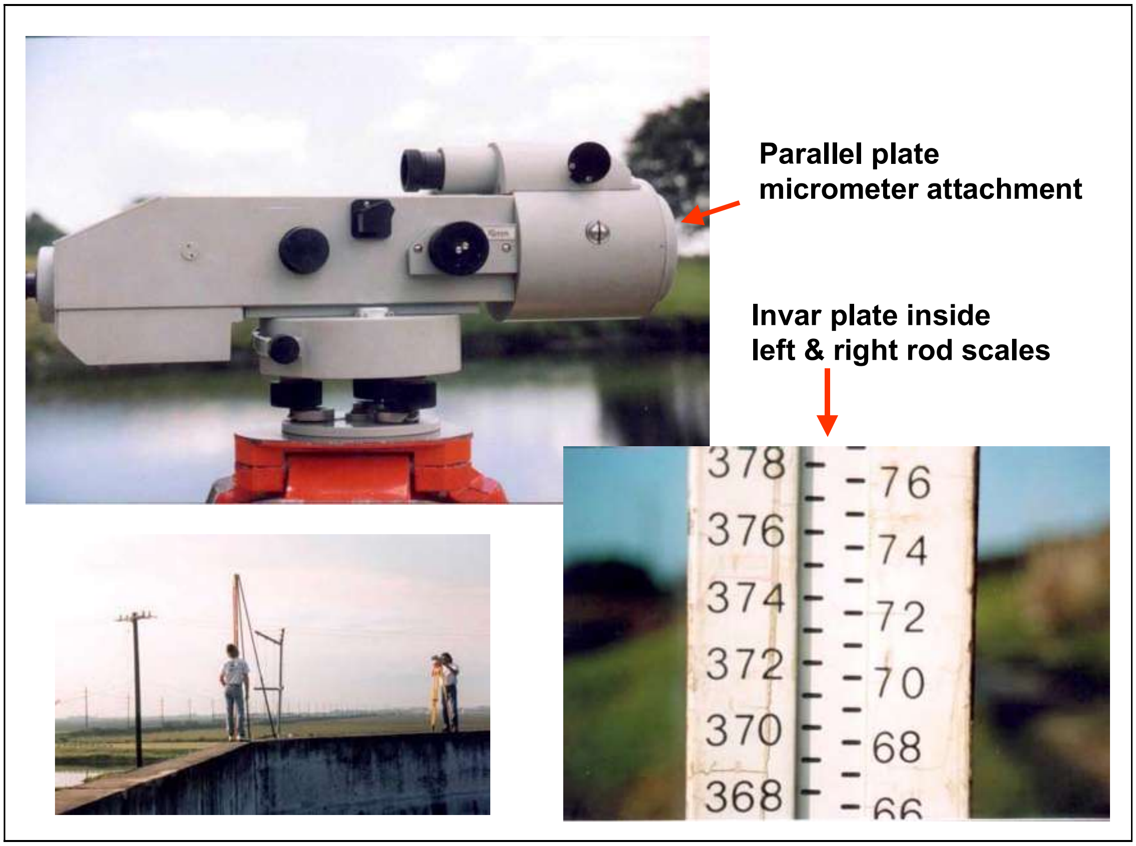


Figure 3-17. Zeiss Ni1 automatic level with parallel plate micrometer attached--precise double-scale Invar rod with constant 3.01550-meter difference in left and right scales (Jacksonville District)

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	LOCATION	ORGANIZATION
Example	Fort Belvoir, Virginia	DMS

OBSERVER	RECORDER	INSTRUMENT	SUN	WIND	WEATHER
SFC Jones	SGT Smith	Wild NA2 - 1234	Warm	Windy	Clear

FROM	TO	DATE (YYYYMMDD)	TIME	LINE OR NET	PAGE NO.	NO. OF PAGES
D2	BASS	2001 07 15	0830-0920	Training 1	no. 2	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]	4310.0			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.

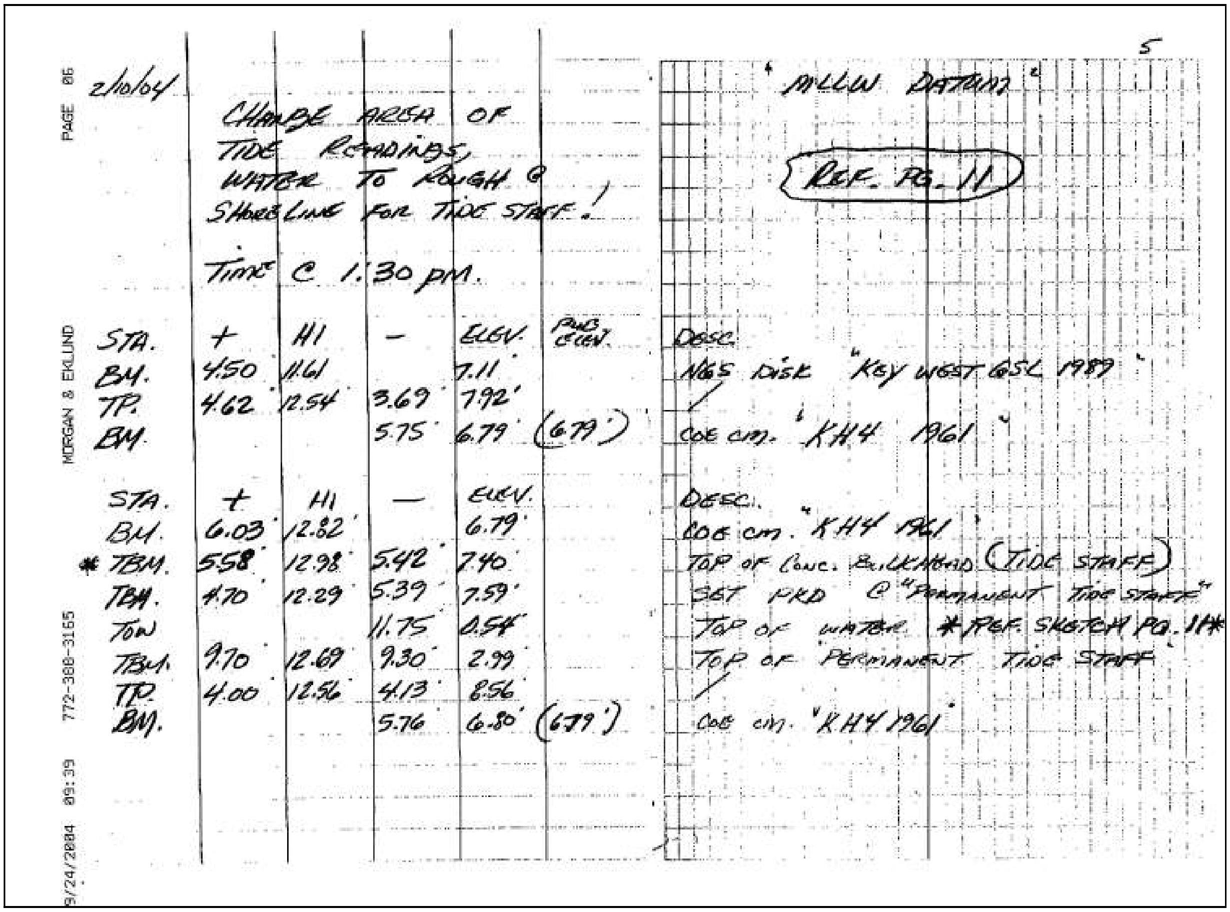
(2) Leveling rods. The rods should be graduated in feet, tenths and hundreds of feet. The Philadelphia rod or its equivalent is acceptable. However, the project specifications will sometimes require that semi-precise rods be used that are graduated on the front in centimeters and on the back in half-foot 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 06

2/10/04 CHANGE AREA OF TIDE READINGS, WATER TO ROUGH @ SHORELINE FOR TIDE STAFF!

TIME @ 1:30 PM.

STA.	+	HI	−	ELEV.	PUB. ELEV.
BM.	4.50	11.61		7.11	
TP.	4.62	12.54	3.69	7.92	
BM.			5.75	6.79	(6.79)

STA.	+	HI	−	ELEV.	
BM.	6.03	12.82		6.79	
* TBM.	5.58	12.98	5.42	7.40	
TBM.	4.70	12.29	5.39	7.59	
TOW			11.75	0.54	
TBM.	9.70	12.69	9.30	2.99	
TP.	4.00	12.56	4.13	8.56	
BM.			5.76	6.80	(6.79)

5

MLLW DATUM

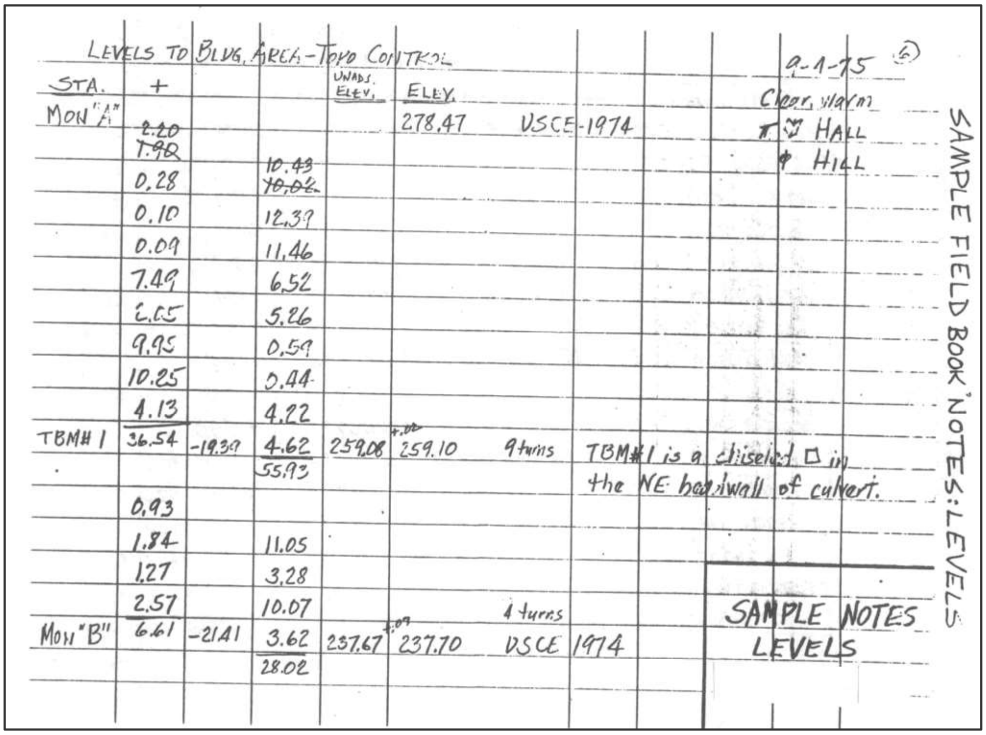
REF. PG. 11

DESC.
NGS DISK "KEY WEST GSL 1989"
COE CM. "KH4 1961"

DESC.
COE CM. "KH4 1961"
TOP OF CONC. BULKHEAD (TIDE STAFF)
SET PKD @ "PERMANENT TIDE STAFF"
TOP OF WATER * REF. SKETCH PG. 11 *
TOP OF PERMANENT TIDE STAFF
COE CM. "KH4 1961"

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

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



LEVELS TO BLDG. AREA – TOPO CONTROL

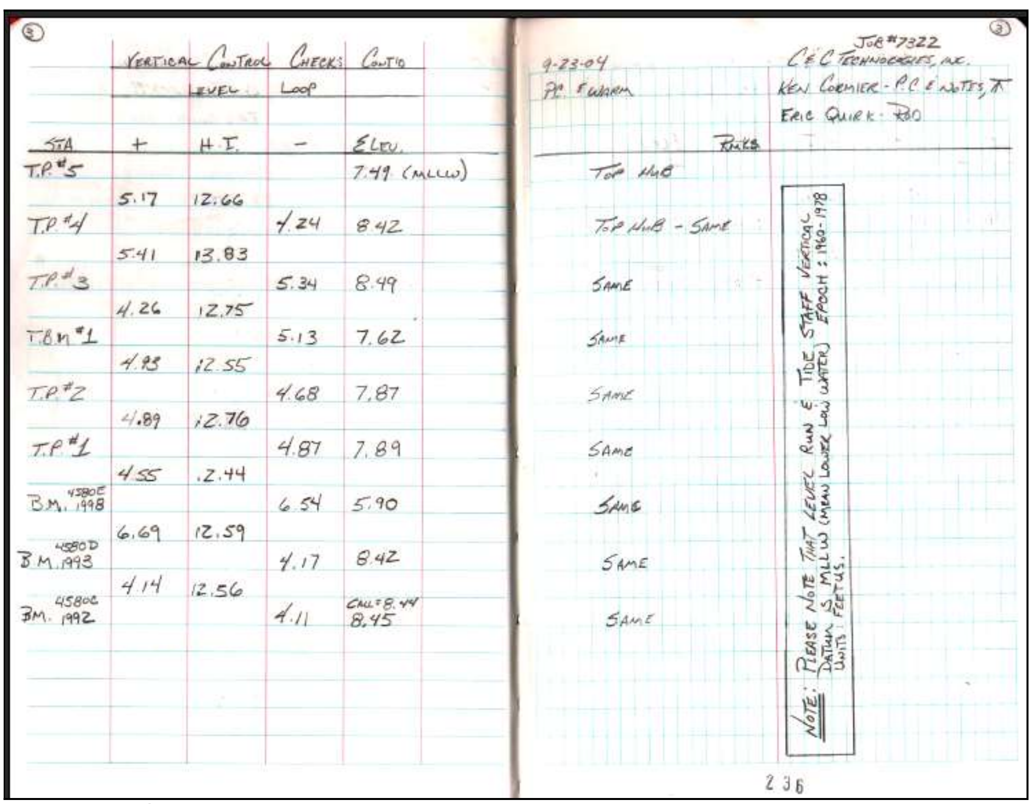
9-1-75
Clear, warm
⊼ ☿ HALL
⌖ HILL

STA.	+			UNADJ. ELEV.	ELEV.		
MON "A"	~~2.20~~ 1.90				278.47	USCE-1974	
	0.28		~~10.43~~ ~~10.02~~				
	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 55.93	259.08	+.02 259.10	9 turns	TBM#1 is a chiseled □ in the NE headwall of culvert.
	0.93						
	1.84		11.05				
	1.27		3.28				
	2.57		10.07			4 turns	
MON "B"	6.61	-21.41	3.62 28.02	237.67	+.03 237.70	USCE 1974	

SAMPLE NOTES LEVELS

SAMPLE FIELD BOOK NOTES: LEVELS

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



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		
TBM#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
PC, WARM

JOB #7322
C&C TECHNOLOGIES, INC.
KEN CORMIER - P.C. & NOTES, ⊼
ERIC QUIRK - ROD

RMKS
TOP HUB
TOP HUB - 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 U.S.

236

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

VERT. DATA TO CHECK TIPS STAFF: 05-27-04

STA.	+	H.I.	−	M.L.L.W. ELEV.	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 OUTSIDE 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)

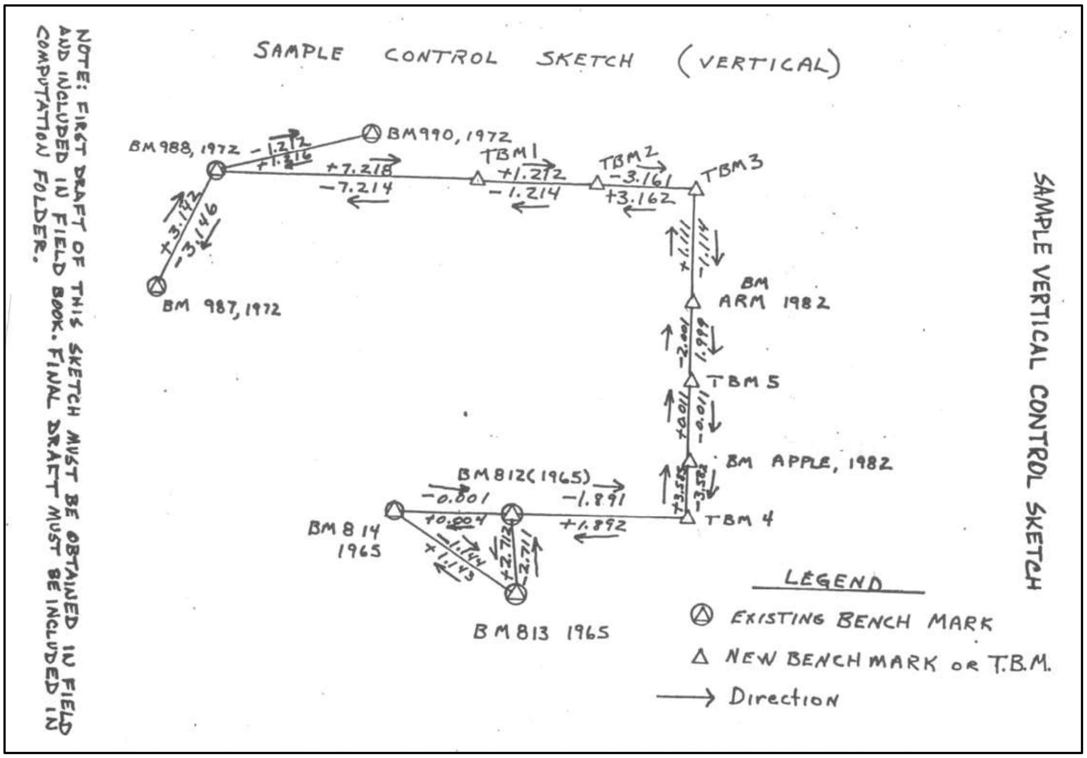


Figure 3-23. Sample vertical control project sketch showing forward and backward level runs for each leg

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.

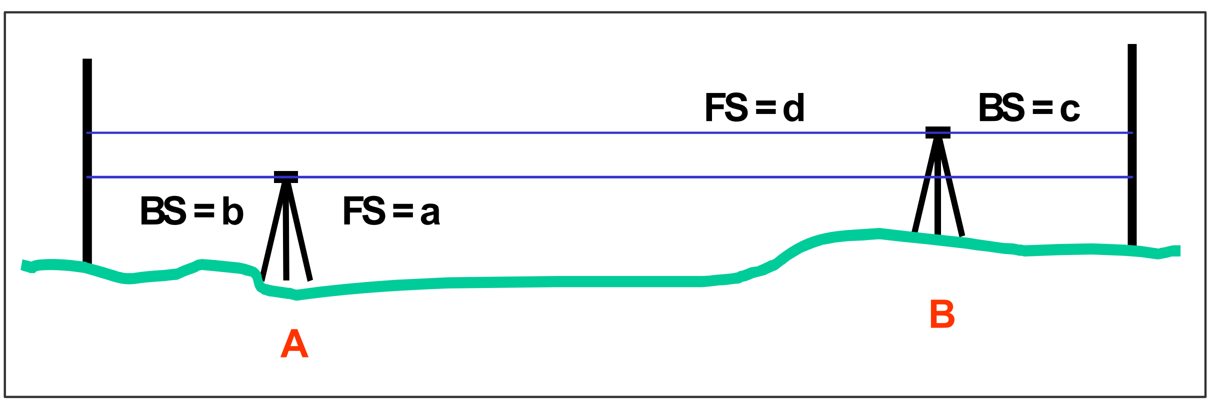


Figure 3-24. C-factor calibration procedure

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

INST OP INT ______

1st COMP INT ______

2nd COMP INT ______

DMS Form 5820-R, JAN 97

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.

Appendix L
Glossary

L-1. Abbreviations and Acronyms

1D .... One Dimensional
2D .... Two-dimensional
2DRMS .... Twice the distance root mean square
3D .... Three-dimensional
A-E .... Architect-Engineer
A/E/C........................... Architect/Engineer/Construction
ACSM.......................... American Congress on Surveying and Mapping
ADA .... Air Defense Artillery
AFB .... Air Force Base
ALTA .... American Land Title Association
AM/FM........................ Automated Mapping/ Facility Mapping
AOC .... Aircraft Obstruction Surveys
ARP .... Antenna Reference Point
ASCE........................... American Society of Civil Engineers
ASPRS......................... American Society for Photogrammetry and Remote Sensing
BFE.............................. Base Flood Elevation
BLM .... Bureau of Land Management
BS .... Backsight
CADD.......................... Computer Aided Drafting and Design
CAiCE .... Computer Aided Civil Engineering
CALTRANS................ California Department of Transportation
CEFMS........................ Corps of Engineers Financial Management System
COGO.......................... Coordinate Geometry
CONUS. .... CONtinental United States
CORPSCON................ CORPS CONvert
CORS .... Continuously Operating Reference Stations
COR............................. Contracting Officer's Representative
DA .... Department of the Army
DE................................ Difference in Elevation
DEM .... Digital Elevation Model
DOD .... Department of Defense
DOT............................. Department of Transportation
DFARS. .... Defense Federal Acquisition Regulation Supplement
DGPS........................... Differential Global Positioning System
DTM .... Digital Terrain Model
EAC............................. Echelons Above Corps
EDM .... Electronic Distance Measurement
EFARS......................... Engineer Federal Acquisition Regulation Supplement
EM............................... Engineer Manual
ERM .... Elevation Reference Mark
ERDC .... Engineer Research and Development Center
E&D. .... Engineering and Design
FA................................ Field Artillery
FAA............................. Federal Aviation Administration
FAC .... Florida Administrative Code

FAR .... Federal Acquisition Regulations
FAR .... Federal Aviation Regulation
FEMA .... Federal Emergency Management Agency
FFP .... Firm Fixed Price
FGCS .... Federal Geodetic Control Subcommittee
FGDC .... Federal Geographic Data Committee
FIRM .... Flood Insurance Rate Map
FIS .... Flood Insurance Study
FLSA .... Fair Labor Standards Act
FM .... Field Manual
FMSFIE........................ Facility Management Standard for Facilities, Infrastructure, and Environment
FOA .... Field Operating Activity
FS.................................. Foresight
G&A .... General and Administrative
GDOP .... Geometric Dilution of Position
GIS................................ Geographic Information System
GPS............................... Global Positioning System
GRS 80 .... Geodetic Reference System of 1980
GS .... General Support
GSA .... General Services Administration
GZD.............................. Grid Zone Designator
HARN .... High Accuracy Regional Networks
HI.................................. Height of Instrument
HDOP .... Horizontal Dilution of Position
HPGN .... High Precision Geodetic Networks
HR .... Height of Reflector
HT................................. Height of Target
HTRW .... Hazardous, Toxic, or radioactive Waste
HQUSACE .... Headquarters, US Army Corps of Engineers
IDC. .... Indefinite Delivery Contract
IERS .... International Earth Rotation Service
IGE .... Independent Government Estimate
IGLD 55 .... International Great Lakes Datum of 1955
IGLD 85 .... International Great Lakes Datum of 1985
ILS .... Instrument Landing System
INT .... Intersection
ITL................................ Information Technology Lab
ITRF .... International Terrestrial Reference Frame
JTR .... Joint Travel Regulation
KO .... Contracting Officer
LCC .... Lambert Conformal Conic
LEC .... Linear Error of Closure
LIDAR.......................... LIght Detection And Ranging
LWRP........................... Low Water Reference Plane
MACOM .... Major Army Command
MDL .... MicroStation Design Language
MGRS........................... Military Grid-Reference System
MHW............................ Mean High Water
MLLW.......................... Mean Lower Low Water
MLRS .... Multiple Launch Rocket System
MLS.............................. Microwave Landing System

MSL.............................. Mean Sea Level
MSL1912...................... Mean Sea Level Datum of 1912
NAD 27 .... North American Datum of 1927
NAD 83 .... North American Datum of 1983
NADCON .... North American Datum Conversion
NAS .... National Airspace System
NAVAID .... Navigation Aid
NAVD 88 .... North American Vertical Datum 1988
NDGPS .... Nationwide Differential GPS
NFIP .... National Flood Insurance Program
NGRS .... National Geodetic Reference System
NGS .... National Geodetic Survey
NGVD 29 .... National Geodetic Vertical Datum 1929
NMAS .... National Map Accuracy Standard
NMP .... National Mapping Program
NOAA .... National Oceanic and Atmospheric Administration
NOS .... National Ocean Service
NSRS .... National Spatial Reference System
NSSDA .... National Standard for Spatial Data Accuracy
NVCN .... National Vertical Control Network
OCONUS .... Outside the Continental United States
OHWM .... Ordinary High Water Mark
OPUS .... On-Line Positioning User Service
OTF .... On-the-Fly
P&S .... Plans and Specifications
PADS .... Position and Azimuth Determination System
PBM .... Permanent Benchmark
PDOP .... Position Dilution of Position
PDSC .... Professional Development Support Center
PI .... Point of Intersection
PLGR .... Precise Lightweight Geodetic Receiver
PM .... Project Manager or Management
POB .... Point of Beginning
POI .... Point on Line
POT .... Point of Tangency
PPRTK .... Post-Processed Real-Time Kinematic
ppm .... Parts per Million
PR&C .... Purchase Request & Commitment
PRIP .... Plant Replacement and Improvement Program
PROSPECT .... Proponent Sponsored Engineer Corps Training
PVT .... Point of Vertical Tangent
QA .... Quality Assurance
QC .... Quality Control
RFP .... Request for Proposal
RMS .... Root mean Square
RMSE .... Root Mean Square Error
RTK .... Real Time Kinematic
SCP .... Survey Control Point
SDSFIE .... Spatial Data Standard for Facilities, Infrastructure, and Environment
SDTS .... Spatial Data Transfer Standard
SI .... International System of Units

SOW .... Scope or Statement of Work
SPCE .... Survey Planning and Coordination Element
SPCS .... State Plane Coordinate System
TA................................ Target Acquisition
TBM .... Temporary Benchmark
TDS .... Tripod Data Systems
TDSE............................ Touchdown Zone Elevation
TEC .... Topographic Engineering Center
TIN .... Triangular Irregular Network
TM .... Transverse Mercator
TGO.............................. Trimble Geomatics Office
TP .... Turning Point
TSC............................... Trimble Survey Controller
US................................. United States
USACE .... US Army Corps of Engineers
USARC......................... US Army Reserve Center
USC&GS .... US Coast & Geodetic Survey
USCG .... US Coast Guard
USFS .... US Forest Service
USGS............................ US Geological Survey
USNAVOCEANO........ US Navy Oceanographic Office
USNG .... US National Grid
UTM .... Universal Transverse Mercator
VDOP .... Vertical Dilution of Position
VERTCON .... VERTical CONversion
VLBI............................. Very-Long-Baseline-Interferometry
WAAS .... Wide Area Augmentation System
WGS 84 .... World Geodetic System of 1984
WRDA.......................... Water Resources Development Act
XREF............................ External Reference
ZD................................ Zenith Distance

L-2. Terms

Absolute or Autonomous GPS
Operation with a single receiver for a desired position. This receiver may be positioned to be stationary over a point. This mode of positioning is the most common military and civil application.

Accuracy
The degree to which an estimated (mean) value is compatible with an expected value. Accuracy implies the estimated value is unbiased.

Adjustment
Adjustment is the process of estimation and minimization of deviations between measurements and a mathematical model.

Altimeter
An instrument that measures elevation differences usually based on atmospheric pressure measurements.

Altitude
The vertical angle between the horizontal plane of the observer and a directional line to the object.

Angle of Depression
A negative altitude.

Angle of Elevation
A positive altitude.

Angular Misclosure
Difference in the actual and theoretical sum of a series of angles.

Archiving
Storing of documents and information.

Astronomical Latitude
Angle between the plumb line and the plane of celestial equator. Also defined as the angle between the plane of the horizon and the axis of rotation of the earth. Astronomical latitude applies only to positions on the earth and is reckoned from the astronomic equator, north and south through 90E. Astronomical latitude is the latitude that results directly from observations of celestial bodies, uncorrected for deflection of the vertical.

Astronomical Longitude
Arbitrarily chosen angle between the plane of the celestial meridian and the plane of an initial meridian. Astronomical longitude is the longitude that results directly from observations on celestial bodies, uncorrected for deflection of the vertical.

Astronomical Triangle
A spherical triangle formed by arcs of great circles connecting the celestial pole, the zenith and a celestial body. The angles of the astronomical triangles are: at the pole, the hour angle; at the celestial body, the parallactic angle; at the zenith, the azimuth angle. The sides are: pole to zenith, the co-latitude; zenith to celestial body, the zenith distance; and celestial body to pole, the polar distance.

Atmospheric Refraction
Refraction of electromagnetic radiation through the atmosphere causing the line-of-sight to deviate from a straight path. Mainly temperature and pressure conditions determine the magnitude and direction of curvature affecting the path of light from a source. Refraction causes the ray to follow a curved path normal the surface gradient.

Azimuth
The horizontal direction of a line clockwise from a reference plane, usually the meridian. Often called forward azimuth to differentiate from back azimuth.

Azimuth Angle
The angle less than 180° between the plane of the celestial meridian and the vertical plane with the observed object, reckoned from the direction of the elevated pole. In astronomic work, the azimuth angle is the spherical angle at the zenith in the astronomical triangle, which is composed of the pole, the zenith and the star. In geodetic work, it is the horizontal angle between the celestial pole and the observed terrestrial object.

Azimuth Closure
Difference in arc-seconds of the measured or adjusted azimuth value with the true or published azimuth value.

Backsight
A sight on a previously established traverse or triangulation station and not the closing sight on the traverse. A reading on a rod held on a point whose elevation has been previously determined.

Barometric Leveling
Determining differences of elevation from measured differences of atmospheric pressure observed with a barometer. If the elevation of one station above a datum is known, the approximate elevations of other station can be determined by barometric leveling. Barometric leveling is widely used in reconnaissance and exploratory surveys.

Baseline
Resultant three-dimensional vector between any two stations with respect to a given coordinate system. The primary reference line in a construction system.

Base net
The primary baseline used for densification of survey stations to form a network.

Base Points
The beginning points for a traverse that will be used in triangulation or trilateration.

Base Control
The horizontal and vertical control points and coordinates used to establish a base network. Base control is determined by field surveys and permanently marked or monumented for further surveys.

Bearing
The direction of a line with respect to the meridian described by degrees, minutes, and seconds within a quadrant of the circle. Bearings are measured clockwise or counterclockwise from north or south, depending on the quadrant.

Benchmark
A permanent material object, natural or artificial, on a marked point of known elevation.

Best Fit
To represent a given set of points by a smooth function, curve, or surface which minimizes the deviations of the fit.

Bipod
A two-legged support structure for an instrument or survey signal at a height convenient for the observer.

Bluebook
Another term for the "FGCS Input Formats and Specifications of the National Geodetic Data Base".

Blunder
A mistake or gross error.

Bureau International de l'Heure
The Bureau was founded in 1919 and its offices since then have been at the Paris Observatory. By an action of the International Astronomical Union, the BIH ceased to exist on 1 January 1988 and a new organization, the International Earth Rotation Service (IERS) was formed to deal with determination of the Earth's rotation.

Cadastral Survey
Relates to land boundaries and subdivisions, and creates units suitable for transfer or to define the limitations of title. The term cadastral survey is now used to designate the surveys of the public lands of the US, including retracement surveys for identification and resurveys for the restoration of property lines; the term can also be applied properly to corresponding surveys outside the public lands, although such surveys are usually termed land surveys through preference.

Calibration
Determining the systematic errors in an instrument by comparing measurements with correct values. The correct value is established either by definition or by measurement with a device that has itself been calibrated or of much higher precision.

Cartesian Coordinates
A system with its origin at the center of the earth and the x and y and z axes in the plane of the equator. Typically, the x-axis passes through the meridian of Greenwich, and the z-axis coincides with the earth's axis of rotation. The three axes are mutually orthogonal and form a right-handed system.

Cartesian System
A coordinate system consisting of axes intersecting at a common point (origin). The coordinate of a point is the orthogonal distance between that point and the hyperplane determined by all axes. A Cartesian coordinate system has all the axes intersecting at right angles, and the system is called a rectangular.

Celestial Equator
A great circle on the celestial sphere with equidistant points from the celestial poles. The plane of the earth's equator, if extended, would coincide with that of the celestial equator.

Celestial pole
A reference point at the point of intersection of an indefinite extension of the earth's axis of rotation and the apparent celestial sphere.

Celestial sphere
An imaginary sphere of infinite radius with the earth as a center. It rotates from east to west on a prolongation of the earth's axis.

Central Meridian
A line of constant longitude at the center of a graticule. The central meridian is used as a base for constructing the other lines of the graticule. The meridian is used as the y-axis in computing tables for a State Plane Coordinate system. That line, on a graticule, which represents a meridian and which is an axis of symmetry.

Chain
Equal to 66 feet or 100 links. The unit of length prescribed by law for the survey of the US public lands. One acre equals 10 square chains.

Chained Traverse
Observations and measurements performed with tape.

Chaining
Measuring distances on the ground with a graduated tape or with a chain.

Chart Datum
Reference surface for soundings on a nautical chart. It is usually taken to correspond to a low water elevation, and its depression below mean sea level is represented by the symbol Z_o. Since 1989, chart datum has been implemented to mean lower low water for all marine waters of the US its territories, Commonwealth of Puerto Rico and Trust Territory of the Pacific Islands.

Chi-square Testing
Non-parametric statistical test used to classify the shape of the distribution of the data.

Chronometer
A portable timekeeper with compensated balance, capable of showing time with extreme precision and accuracy.

Circle Position
A prescribed setting (reading) of the horizontal circle of a direction theodolite, to be used for the observation on the initial station of a series of stations that are to be observed.

Circuit Closure
Difference between measured or adjusted value and the true or published value.

Clarke 1866 Ellipsoid
The reference ellipsoid used for the NAD 27 horizontal datum. It is a non-geocentric ellipsoid formerly used for mapping in North America.

Closed Traverse
Starts and ends at the same point or at stations whose positions have been determined by other surveys.

Collimation
A physical alignment of a survey target or antenna over a mark or to a reference line.

Collimation Error
The angle between the actual line of sight through an optical instrument and an alignment.

Compass Rule
The correction applied to the departure (or latitude) of any course in a traverse has the same ratio to the total misclosure in departure (or latitude) as the length of the course has to the total length of the traverse.

Confidence Level

Statistical probability (in percent) based on the standard deviation or standard error associated with the normal probability density function. The confidence level is assigned according to an expansion factor multiplied by the magnitude of one standard error. The expansion factor is based on values found in probability tables at a chosen level of significance.

Conformal
Map projection that preserves shape.

Contour
An imaginary line on the ground with all points at the same elevation above or below a specified reference surface.

Control
Data used in geodesy and cartography to determine the positions and elevations of points on the earth's surface or on a cartographic representation of that surface. A collective term for a system of marks or objects on the earth or on a map or a photograph whose positions or elevation are determined.

Control Densification
Addition of control throughout a region or network.

Control Monuments
Existing local control or benchmarks that may consist of any Federal, state, local or private agency points.

Control Point
A point with assigned coordinates is sometimes used as a synonym for control station. However, a control point need not be realized by a marker on the ground.

Control Survey
A survey which provides coordinates (horizontal or vertical) of points to which supplementary surveys are adjusted.

Control Traverse
A survey traverse made to establish control.

Conventional Terrestrial Pole (CTP)
The origin of the WGS 84 Cartesian system is the earth's center of mass. The Z-axis is parallel to the direction of the CTP for polar motion, as defined by the Bureau of International de l'Heure (BIH), and equal to the rotation axis of the WGS 84 ellipsoid. The X-axis is the intersection of the WGS 84 reference meridian plane and the CTP's equator, the reference meridian being parallel to the zero meridian defined by the BIH and equal to the X-axis of the WGS 84 ellipsoid. The Y-axis completes a right-handed, earth-centered, earth-fixed orthogonal coordinate system, measured in the plane of the CTP equator 90 degrees east of the X-axis and equal to the Y-axis of the WGS 84 ellipsoid.

Coordinate Transformation
A mathematical process for obtaining a modified set of coordinates through some combination of rotation of coordinate axes at their point of origin, change of scale along coordinate axes, or translation through space

CORPSCON
(Corps Convert) Software package (based on NADCON) capable of performing coordinate transformations between NAD 83 and NAD 27 datums.

Crandall Method
Traverse misclosure in azimuth or angle is first distributed in equal portions to all the measured angles. The adjusted angles are then held fixed and all remaining coordinate corrections distributed among the distance measurements.

Cross sections
A survey line run perpendicular to the alignment of a project, channel or structure.

Curvature
The rate at which a curve deviates from a straight line. The parametric vector described by dt/ds, where t is the vector tangent to a curve and s is the distance along that curve.

Datum
Any numerical or geometrical quantity or set of such quantities which serve as a reference or base for other quantities.

Declination
The angle, at the center of the celestial sphere, between the plane of the celestial equator and a line from the center to the point of interest (on a celestial body).

Deflection of the Vertical
The spatial angular difference between the upward direction of a plumb line and the normal to the reference ellipsoid. Often expressed in two orthogonal components in the meridian and the prime vertical directions.

Deflection Traverse
Direction of each course measured as an angle from the direction of the preceding course.

Deformation Monitoring
Observing the movement and condition of structures by describing and modeling its change in shape.

Departure
The orthogonal projection of a line onto an east-west axis of reference. The departure of a line is the difference of the meridional distances or longitudes of the ends of the line.

Differential GPS
Process of measuring the differences in coordinates between two receiver points, each of which is simultaneously observing/measuring satellite code ranges and/or carrier phases from the NAVSTAR GPS constellation. Relative positioning with GPS can be performed by a static or kinematic modes.

Differential Leveling
The process of measuring the difference of elevation between any two points by spirit leveling.

Direction
The angle between a line or plane and an arbitrarily chosen reference line or plane. At a triangulation station, observed horizontal angles are referred to a common reference line and termed horizontal direction. A line, real or imaginary, pointing away from some specified point or locality toward another point. Direction has two meanings: that of a numerical value and that of a pointing line.

Direct Leveling
The determination of differences of elevation through a continuous series of short horizontal lines. Vertical distances from these lines to adjacent ground marks are determined by direct observations on graduated rods with a leveling instrument equipped with a spirit level.

Distance Angle
An angle in a triangle opposite a side used as a base in the solution of the triangle, or a side whose length is to be computed.

Dumpy Level
The telescope permanently attached to the leveling base, either rigidly to by a hinge that can be manipulated by a micrometer screw.

Earth-Centered Ellipsoid
Center at the Earth's center of mass and minor semi-axis coincident with the Earth's axis of rotation.

Easting
The distance eastward (positive) or westward (negative) of a point from a particular meridian taken as reference.

Eccentricity
The ratio of the distance from the center of an ellipse to its focus on the major semi-axis.

Electronic Distance Measurement (EDM)
Timing or phase comparison of electro-magnetic signal to determine an interferometric distance.

Elevation
The height of an object above some reference datum.

Ellipsoid
Formed by revolving an ellipse about its minor semi-axis. The most commonly used reference ellipsoids in North America are: Clarke 1866, Geodetic Reference System of 1980 (GRS 80), World Geodetic System of 1972 (WGS 72) and World Geodetic System of 1984 (WGS 84).

Ellipsoid height
The magnitude h of a point above or below the reference ellipsoid measured along the normal to the ellipsoid surface.

Error
The difference between the measured value of a quantity and the theoretical or defined value of that quantity.

Error Ellipse
An elliptically shaped region with dimensions corresponding to a certain probability at a given confidence level.

Error of Closure
Difference in the measured and predicted value of the circuit along the perimeter of a geometric figure.

Finite Element Method
Obtaining an approximate solution to a problem for which the governing differential equations and boundary conditions are known. The method divides the region of interest into numerous, interconnected sub-regions (finite elements) over which simple, approximating functions are used to represent the unknown quantities.

Fixed Elevation
Adopted as a result of tide observations or previous adjustment of spirit leveling, and which is held at its accepted value in any subsequent adjustment.

Foresight
An observation to the next instrument station. The reading on a rod that is held at a point whose elevation is to be determined.

Frequency
The number of complete cycles per second existing in any form of wave motion.
Geodesic Line
Shortest distance between any two points on any mathematically defined surface.

Geodesy
Determination of the time-varying size and figure of the earth by such direct measurements as triangulation, leveling and gravimetric observations.

Geodetic Control
Established and adjusted horizontal and/or vertical control in which the shape and size of the earth have been considered in position computations.

Geodetic Coordinates
Angular latitudinal and longitudinal coordinates defined with respect to a reference ellipsoid.

Geodetic Height
See Ellipsoid height.

Geodetic Latitude
The angle which the normal at a point on the reference ellipsoid makes with the plane of the equator.

Geodetic Leveling
The observation of the differences in elevation by means of a continuous series of short horizontal lines of sight.

Geodetic Longitude
The angle subtended at the pole between the plane of the geodetic meridian and the plane of a reference meridian (Greenwich).

Geodetic North
Direction tangent to a meridian pointing toward the pole defining astronomic north, also called true north.

Geodetic Reference System of 1980
Reference ellipsoid used to establish the NAD 83 system of geodetic coordinates.

Geoid
An equipotential surface of the gravity field approximating the earth's surface and corresponding with mean sea level in the oceans and its extension through the continents.

GPS (Global Positioning System)
DoD satellite constellation providing range, time, and position information through a GPS receiver system.

Gravimeter
Instrument for measuring changes in gravity between two points.

Gravity
Combined acceleration potential of an object due to gravitation and centrifugal forces.

Greenwich Meridian
The astronomic meridian through the center of the Airy transit instrument of the Greenwich Observatory, Greenwich, England. By international agreement in 1884, the Greenwich meridian was adopted as the meridian from which all longitudes, worldwide, would be calculated.

Grid Azimuth
The angle in the plane of projection between a straight line and the line (y-axis) in a plane rectangular coordinate system representing the central meridian. While essentially a map-related quantity, a grid azimuth may, by mathematical processes, be transformed into a survey- related or ground-related quantity.

Grid Inverse
The computation of length and azimuth from coordinates on a grid.

Grid Meridian
Line parallel to the line representing the central meridian or y-axis of a grid on a map. The map line parallel to the line representing the y-axis or central meridian in a rectangular coordinate system.

Gunter's Chain
A measuring device once used in land surveying. It was composed of 100 metallic links fastened together with rings. The total length of the chain is 66 feet. Also called a four-pole chain.

Gyrotheodolite
A gyroscopic device used to measure azimuth that is built-in or attached to a theodolite.

Histogram
A graphical representation of relative frequency of an outcome partitioned by class interval. The frequency of occurrence is indicated by the height of a rectangle whose base is proportional to the class interval.

Horizontal Control
Determines horizontal positions with respect to parallels and meridians or to other lines of reference.

Hour Circle
Any great circle on the celestial sphere whose plane is perpendicular to the plane of the celestial equator.

Index Error
A systematic error caused by deviation of an index mark or zero mark on an instrument having a scale or vernier, so that the instrument gives a non-zero reading when it should give a reading of zero. The distance error from the foot of a leveling rod to the nominal origin (theoretical zero) of the scale.

Indirect Leveling
The determination of differences of elevation from vertical angles and horizontal distances.

Interior Angle
An angle between adjacent sides of a closed figure and lying on the inside of the figure. The three angles within a triangle are interior angles.

International Foot
Defined by the ratio 30.48/100 meters.

International System of Units (SI)
A self-consistent system of units adopted by the general Conference on Weights and Measures in 1960 as a modification of the then-existing metric system.

Interpolation Method
Determination of a intermediate value between given values using a known or assumed rate of change of the values between the given values.

Intersection
Determining the horizontal position of a point by observations from two or more points of known position. Thus measuring directions or distances that intersect at the station being located. A station whose horizontal position is located by intersection is known as an intersection station.

Intervisibility
When two stations are visible to each other in a survey net.

Invar
An alloy of iron containing nickel, and small amounts of chromium to increase hardness, manganese to facilitate drawing, and carbon to raise the elastic limit, and having a very low coefficient of thermal expansion (about 1/25 that of steel).

Isogonic Chart
A system of isogonic lines, each for a different value of the magnetic declination.

Isogonic Line
A line drawn on a chart or map and connecting all points representing points on the earth having equal magnetic declination at a given time.

Laplace Azimuth
A geodetic azimuth derived from an astronomic azimuth by use of the Laplace equation.

Laplace Condition
Arises from the fact that a deflection of the vertical in the plane of the prime vertical will give a difference between astronomic and geodetic longitude and between astronomic and geodetic azimuth. Conversely, the observed differences between astronomic and geodetic values of the longitude and of the azimuth may both be used to determine the deflection in the plane of the prime vertical.

Laplace Equation
Expresses the relationship between astronomic and geodetic azimuths in terms of astronomic and geodetic longitudes and geodetic latitude.

Laplace Station
A triangulation or traverse station at which a Laplace azimuth is determined. At a Laplace station both astronomic longitude and astronomic azimuth are determined.

Least Count
The finest reading that can be made directly (without estimation) from a vernier or micrometer.

Least Squares Adjustment
The adjustment of the values of either the measured angles or the measured distances in a traverse using the condition that the sum of the squares of the residuals is a minimum.

Level
Any device sensitive to the direction of gravity and used to indicate directions perpendicular to that of gravity at a point.

Level Datum
A level surface to which elevations are referred. The generally adopted level datum for leveling in the US is mean sea level. For local surveys, an arbitrary level datum is often adopted and defined in terms of an assumed elevation for some physical mark.

Level Net
Lines of spirit leveling connected together to form a system of loops or circuits extending over an area.

Line of Sight
The line extending from an instrument along which distant objects are seen, when viewed with a telescope or other sighting device.

Local Coordinate System
Where the coordinate system origin is assigned arbitrary values and is within the region being surveyed and used principally for points within that region.

Local Datum
Defines a coordinate system that is used only over a region of very limited extent.

Loop Traverse
A closed traverse that starts and ends at the same station. A pattern of measurements in the field, so that the final measurement is made at the same place as the first measurement.

Magnetic Bearing
The angle with respect to magnetic north or magnetic south stated as east or west of the magnetic meridian.

Magnetic Meridian
The vertical plane through the magnetic pole including the direction, at any point, of the horizontal component of the Earth's magnetic field.

Major Semi-Axis
The line from the center of an ellipse to the extremity of the longest diameter. The term is also used to mean the length of the line.

Map
A conventional representation, usually on a plane surface and at an established scale, of the physical features (natural, artificial, or both) of a part or whole of the Earth's surface by means of signs and symbols and with the means of orientation indicated.

Map Accuracy
The accuracy with which a map represents. Three types of error commonly occur on maps: errors of representation, which occur because conventional signs must be used to represent natural or man-made features such as forests, buildings and cities; errors of identification, which occur because a non-existent feature is shown or is misidentified; and errors of position, which occur when an object is shown in the wrong position. Errors of position are commonly classified into two types: errors of horizontal location and errors of elevation. A third type, often neglected, is errors of orientation.

Map Scale
The ratio of a specified distance on a map to the corresponding distance in the mapped object.

Mean Angle
Average value of the angles.

Mean Lower Low Water (MLLW)
The average height of all lower low waters recorded over a 19-year period.
Mean Sea Level Datum
Adopted as a standard datum for heights or elevations. The Sea Level Datum of 1929, the current standard for geodetic leveling in the United States, is based on tidal observations over a number of years at various tide stations along the coasts.

Metric Unit
Belonging to or derived from the SI system of units.

Micrometer
In general, any instrument for measuring small distances very accurately. In astronomy and geodesy, a device, for attachment to a telescope or microscope, consisting of a mark moved across the field of view by a screw connected to a graduated drum and vernier. If the mark is a hair-like filament, the micrometer is called a filar micrometer.

Minor Semi-Axis
The line from the center of an ellipse to the extremity of the shortest diameter. I.e., one of the two shortest lines from the center to the ellipse. The term is also used to mean the length of the line.

Misclosure
The difference between a computed and measured value.

Monument
A physical object used as an indication of the position on the ground of a survey station.

NADCON
The National Geodetic Survey developed the conversion program NADCON (North American Datum Conversion) to convert to and from North American Datum of 1983. The technique used is based on a bi-harmonic equation classically used to model plate deflections. NADCON works exclusively in geographical coordinates (latitude/longitude).

Nadir
The point directly beneath the instrument and directly opposite to the zenith or the lowest point.

National Geodetic Vertical Datum 1929
Formerly adopted as the standard geodetic datum for heights, based on an adjustment holding 26 primary tide stations in North America fixed.

National Map Accuracy Standards
Specifications of the accuracy required of topographic maps published by the US at various scales.

National Tidal Datum Epoch
A period of 19 years adopted by the National Ocean Survey as the period over which observations of tides are to be taken and reduced to average values for tidal datums.

Network
Interconnected system of surveyed points.

Non-SI units
Units of measurement not associated with International System of Units (SI).

North American Datum of 1927
Formerly adopted as the standard geodetic datum for horizontal positioning. Based on the Clarke ellipsoid of 1866, the geodetic positions of this system are derived from a readjustment of survey observations throughout North America.

North American Datum of 1983
Adopted as the standard geodetic datum for horizontal positioning. Based on the Geodetic Reference System of 1980, the geodetic positions of this system are derived from a readjustment of survey observations throughout North America.

North American Vertical Datum of 1988
Adopted as the standard geodetic datum for heights.

Northing
A linear distance, in the coordinate system of a map grid, northwards from the east-west line through the origin (or false origin).

Open Traverse
Begins from a station of known or adopted position, but does not end upon such a station.

Optical Micrometer
Consists of a prism or lens placed in the path of light entering a telescope and rotatable, by means of a graduated linkage, about a horizontal axis perpendicular to the optical axis of the telescope axis. Also called an optical-mechanical compensator. The device is usually placed in front of the objective of a telescope, but may be placed immediately after it. The parallel-plate optical micrometer is the form usually found in leveling instruments.

Optical Plummet
A small telescope having a 90° bend in its optical axis and attached to an instrument in such a way that the line of sight proceeds horizontally from the eyepiece to a point on the vertical axis of the instrument and from that point vertically downwards. In use, the observer, looking into the plummet, brings a point on the instrument vertically above a specified point (usually a geodetic or other mark) below it.

Order of Accuracy
Defines the general accuracy of the measurements made in a survey. The order of accuracy of surveys are divided into four classes labeled: First Order, Second Order, Third Order and Fourth or lower order.

Origin
That point in a coordinate system which has defined initial coordinates and not coordinates determined by measurement. This point is usually given the coordinates (0,0) in a coordinate system in the plane and (0,0,0) in a coordinate system in space.

Orthometric Height
The elevation H of a point above or below the geoid.

Parallax
The apparent displacement of the position of a body, with respect to a reference point or system, caused by a shift in the point of observation.

Philadelphia Leveling Rod
Having a target but with graduations so styled that the rod may also be used as a self-reading leveling rod. Also called a Philadelphia rod. If a length greater than 7 feet is needed, the target is clamped at 7 feet and raised by extending the rod. When the target is used, the rod is read by vernier to 0.001 foot. When the rod is used as a self-reading leveling rod, the rod is read to 0.005 foot.

Photogrammetry
Deducing the physical dimensions of objects from measurements on photographs of the objects.

Picture Point
A terrain feature easily identified on an aerial photograph and whose horizontal or vertical position or both have been determined by survey measurements. Picture points are marked on the aerial photographs by the surveyor, and are used by the photomapper.

Planetable
A field device for plotting the lines of a survey directly from observations. It consists essentially of a drawing board mounted on a tripod, with a leveling device designed as part of the board and tripod.

Planimetric Feature
Item detailed on a planimetric map.

Plumb Line
The direction normal to the geopotential field. The continuous curve to which the gradient of gravity is everywhere tangential.

Positional Error
The amount by which the actual location of a cartographic feature fails to agree with the feature's true position.

Post-Processed Real-Time Kinematic GPS
GPS carrier phase positioning performed without real-time data link and solution.

Precision
The amount by which a measurement deviates from its mean.

Prime Meridian
The meridian of longitude 0°, used as the origin for measurement of longitude. The meridian of Greenwich, England, is almost universally used for this purpose.

Prime Vertical
The vertical circle through the east and west points of the horizon. It may be true, magnetic, compass or grid depending upon which east or west points are involved.

Project Control
Control used for a specific project.

Project Datum
Datum used for a specific project.

Projection
A set of functions, or the corresponding geometric constructions, relating points on one surface to points on another surface. A projection requires every point on the first surface to correspond one-to-one to points on the second surface.

Quadrangle
Consisting of four specified points and the lines or line segments on which they lie. The quadrangle and the quadrilateral differ in that the quadrangle is defined by four specified angle points, the quadrilateral by four specified lines or line-segments.
Random Error
Randomly distributed deviations from the mean value.

Range Pole
A simple rod fitted with a sharp-pointed, shoe of steel and usually painted alternately in red and white bands at 1-foot intervals.

Readings
The observed value obtained by noting and/or recording scales.

Real-time
An event or measurement reported or recorded at the same time as the event is occurring through the absence of delay in getting, sending and receiving data.

Real-Time Kinematic GPS
GPS carrier phase processing and positioning in real-time.

Reciprocal Leveling
Measuring vertical angles or making rod readings from two instrument positions for the purpose of compensating for the effects of refraction.

Rectangular Coordinate Systems
Coordinates on any system in which the axes of reference intersect at right angles.

Redundant Measurements
Taking more measurements than are minimally required for a unique solution.

Reference Meridian, True
Based on the astronomical meridian.

Reference Meridian, Magnetic
Based on the magnetic pole.

Reference Point
Used as an origin from which measurements are taken or to which measurements are referred.

Refraction
The bending of rays by the substance through which the rays pass. The amount and direction of bending are determined by its refractive index.

Relative Accuracy
Indicated by the dimensions of the relative confidence ellipse between two points. A quantity expressing the effect of random errors on the location of one point or feature with respect to another.

Repeating Theodolite
Designed so that the sum of successive measurements of an angle can be read directly on the graduated horizontal circle.

Resection
Determining the location of a point by extending lines of known direction to two other known points.

Sexagesimal System
Notation by increments of 60. As the division of the circle into 360°, each degree into 60 minutes, and each minute into 60 seconds.

Set-up
In general, the situation in which a surveying instrument is in position at a point from which observations are made.

Spheroid
Used as a synonym for ellipsoid.

Spirit Level
A closed glass tube (vial) of circular cross section. Its center line forms a circular arc with precise form and filled with ether or liquid of low viscosity, with enough free space left for a bubble of air or gas.

Stadia Constant
The sum of the focal length of a telescope and the distance from the vertical axis of the instrument on which the telescope is mounted to the center of the objective lens-system.

Stadia Traverse
Distances are determined using a stadia rod. A stadia traverse is suited to regions of moderate relief with an adequate network of roads. If done carefully, such a traverse can establish elevations accurate enough for compiling maps with any contour interval now standard.

Standard Error
The standard deviation of the errors associated with physical measurements of an unknown quantity, or statistical estimates of an unknown quantity or of a random variable.

Systematic Error
Errors that affect the position (bias) of the mean. Systematic errors are due to unmodeled affects on the measurements that have a constant or systematic value.

State Plane Coordinate System (SPCS)
A planar reference coordinate system used in the United States.

Strength of Figure
A number relating the precision in positioning with the geometry with which measurements are made.

Subtense Bar
A bar with two marks at a fixed, known distance apart used for determining the horizontal distance from an observer by means of the measuring the angle subtended at the observer between the marks.

Taping
Measuring a distance on the using a surveyor's tape.
Three-wire Leveling
The scale on the leveling rod is read at each of the three lines and the average is used for the final result.

Topographic Map
A map showing the horizontal and vertical locations of the natural and man-made features represented and the projected elevations of the surroundings.

Transformation
Converting a position from one coordinate system to another.

Transit
The apparent passage of a star or other celestial body across a defined line of the celestial sphere.

Transit Rule
The correction to be applied to the departure (or latitude) of any course has the same ratio to the total misclosure in departure (or latitude) as the departure (latitude) of the course has to the arithmetical sum of all the departures (latitudes) in the traverse. The transit rule is often used when it is believed that the misclosure is caused less by errors in the measured angles than by errors in the measured distances.

Transverse Mercator Projection
Mercator map projection calculated for a cylinder with axis in the equatorial plane.

Traverse
A sequence of points along which surveying measurements are made.

Triangulation
Determination of positions in a network by the measurement of angles between stations.

tribrach
The three-armed base, of a surveying instrument, in which the foot screws used in leveling the instrument are placed at the ends of the arms. Also called a leveling base or leveling head.

Trigonometric heighting
The trigonometric determination of differences of elevation from observed vertical angles and measured distances.

Trilateration
Determination of positions in a network by the measurement of distances between stations using the intersection of two or more distances to a point.

Universal Transverse Mercator
A worldwide metric military coordinate system.

US Coast & Geodetic Survey (USC&GS)
Now known as National Ocean Service (NOS).

US Survey Foot
The unit of length defined by 1200/3937 m

Variance-Covariance Matrix
A matrix whose elements along the main diagonal are called the variances of the corresponding variables; the elements off the main diagonal are called the covariances.

Vernier
An auxiliary scale used in reading a primary scale. The total length of a given number of divisions on a vernier is equal to the total length of one more or one less than the same number of divisions on the primary scaled.

VERTCON
Acronym for vertical datum conversion. VERTCON is the computer software that converts orthometric heights between NGVD 29 to NAVD 88.

Vertical Angle
An angle in a vertical plane either in elevation or depression from the horizontal.

Vertical Circle
A graduated scale mounted on an instrument used to measure vertical angles.

Vertical Datum
Any level surface used as a reference for elevations. Although a level surface is not a plane, the vertical datum is frequently referred to as the datum plane.

World Geodetic System of 1984
Adopted as the standard geodetic datum for GPS positioning. Based on the World Geodetic System reference ellipsoid.

Wye Level
Having the telescope and attached spirit level supported in wyes (Y's) in which it can be rotated about its longitudinal axis (collimation axis) and from which it can be lifted and reversed, end for end. Also called a Y-level and wye-type leveling instrument.

Zenith
The point above the instrument where an extension of a plumb (vertical) line at the observer's position intersects the celestial sphere.

Zenith Angle
Measured in a positive direction downwards from the observer's zenith to the observed target.

Zenith Distance
The complement of the altitude, the angular distance from the zenith of the celestial body measured along a vertical circle.