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	Engineering and Design DEFORMATION MONITORING AND CONTROL SURVEYING	
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US Army Corps
of Engineers

ENGINEERING AND DESIGN

Deformation Monitoring and Control Surveying

ENGINEER MANUAL

	DEPARTMENT OF THE ARMY	EM 1110-1-1004
	U.S. Army Corps of Engineers	
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Manual
No. 1110-1-1004 — 31 October 1994

Engineering and Design
DEFORMATION MONITORING AND CONTROL SURVEYING

1. Purpose. This manual provides technical specifications and procedural guidance for control, geodetic and precise structural deformation surveying. It is intended for use by engineering, topographic, and construction surveyors performing control or deformation surveys for civil works, military construction, and environmental restoration projects. Procedural and quality control standards are defined to establish uniformity in control survey performance and contract administration.

2. Applicability. This manual applies to HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having responsibility for the planning, engineering and design, operations, maintenance, construction, and related real estate and regulatory functions of civil works, military construction, and environmental restoration projects. It applies to control surveys performed by both hired-labor forces and contracted survey forces. It is also applicable to surveys performed or procured by local interest groups under various cooperative or cost-sharing agreements.

3. General. Control survey techniques can be used to formulate accurate, three-dimensional point positions. Positions obtained through control surveying may be used to provide the primary reference control monument locations for engineering and construction projects, from which detailed site plan topographic mapping, boundary demarcation, and construction alignment work can be performed. Positions obtained through control surveying have application in the continuous positioning of marine construction vessels, such as dredges and survey boats. Also, various control survey techniques can be used to effectively and efficiently monitor and evaluate large structures, such as locks and dams.

4. Accuracy. The accuracy of USACE surveying measurements should be consistent with the purpose of the survey. When evaluating the technique to be used and accuracies desired, the surveyor must evaluate the limits of the errors of the equipment involved, the procedures to be followed, and effects of error propagation. These evaluations should be firmly based on past experience or written guidance. It is key to remember in this evaluation that the best survey is the one that provides the data at the required accuracy levels without wasting manpower, time, and money.

FOR THE COMMANDER:

ROBERT L. VANANTWERP
Colonel, Corps of Engineers
Chief of Staff

ETL 1110-1-147 is superseded and incorporated in this manual.

CECW-EP

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Chapter 1
Introduction

1-1. Purpose

This manual provides technical specifications and procedural guidance for control, geodetic and precise structural deformation surveying. It is intended for use by engineering, topographic, and construction surveyors performing control or deformation surveys for civil works, military construction, and environmental restoration projects. Procedural and quality control standards are defined to establish uniformity in control survey performance and contract administration.

1-2. Applicability

This manual applies to HQUSACE elements, major subordinate commands (MSC), districts, laboratories, and field operating activities (FOA) having responsibility for the planning, engineering and design, operations, maintenance, construction, and related real estate and regulatory functions of civil works, military construction, and environmental restoration projects. It applies to control surveys performed by both hired-labor forces and contracted survey forces. It is also applicable to surveys performed or procured by local interest groups under various cooperative or cost-sharing agreements.

1-3. References

Required and related publications are listed in Appendix A.

1-4. Explanation of Abbreviations and Terms

Control surveying terms and abbreviations used in this manual are explained in the Glossary (Appendix B).

1-5. Background

a. A control survey consists of establishing the horizontal and vertical positions of points for the control of a project site, map, or study area. A geodetic control survey takes into consideration the size and shape of the earth; it implies a reference ellipsoid which represents the geoid and the vertical control datums. A structural deformation survey involves the accurate measurement of short-term and long-term structural deflections or deformations. Geodetic survey techniques are used in performing deformation measurements.

b. Conventional survey techniques are those which use traditional ground survey instruments. They do not include Global Positioning System (GPS) control survey techniques. Typically, conventional survey techniques include traverse, triangulation, trilateration, and differential leveling.

c. Control survey techniques can be used to formulate accurate, three-dimensional point positions. Positions obtained through control surveying may be used to provide the primary reference control monument locations for engineering and construction projects, from which detailed site plan topographic mapping, boundary demarcation, and construction alignment work can be performed. Positions obtained through control surveying have application in the continuous positioning of marine construction vessels, such as dredges and survey boats. Also, various control survey techniques can be used to effectively and efficiently monitor and evaluate large structures, such as locks and dams.

1-6. Scope of Manual

This manual covers the use of control survey techniques (horizontal and vertical) for establishing and/or extending project construction or boundary control. Azimuth determination procedures, data reduction and adjustment methods, and control surveying techniques are outlined. A primary emphasis of this manual centers on the technical procedures for performing precise surveys in support of structural deformation monitoring.

a. The manual is intended to be a reference guide for control and deformation surveying, whether performed by in-house hired-labor forces, contracted forces, or combinations thereof. General planning criteria, field and office execution procedures, and required accuracy specifications for performing control surveys are provided. Accuracy specifications, procedural criteria, and quality control requirements contained in this manual should be directly referenced in the scopes of work for Architect-Engineer (A-E) survey services or other third-party survey services. This ensures that standardized procedures are followed by both hired-labor and contract service sources.

b. The specific project control survey procedures and standards are designed to achieve adequate and economical results to support USACE engineering and construction activities. Thus, the primary emphasis of the manual centers on performing Second- and Third-Order accuracy surveys. This accuracy level will provide adequate reference control from which supplemental real estate, engineering, construction layout surveying, and site plan topographic mapping work may be performed. Therefore, the survey criteria, given in this manual, are not intended to meet the Federal Geodetic Control Committee (FGCC) (now the Federal Geodetic Control Subcommittee (FGCS)) standards and specifications required for densifying the National Geodetic Reference System (NGRS). However, following the methods and procedures given in this manual will yield results equal to or exceeding FGCS Second-Order relative accuracy criteria. Second-Order accuracy is sufficient for USACE engineering and construction work.

c. When a project requires NGRS densification, or such densification is a desirable by-product and is economically justified, USACE Commands should conform to the more rigorous FGCS "Standards and Specifications for Geodetic Control Networks." This includes related automated data recording, submittal, and project review requirements mandated by FGCS and the National Geodetic Survey (NGS). Details outlining the proposed use of control surveying techniques, including specific requirements for connections to the NGRS, shall be included in the descriptions of surveying and mapping activities contained in project authorization documents.

d. This manual does not cover the theory and physical concepts of GPS survey techniques. For further specific guidance on all aspects of GPS surveying, the user should consult EM 1110-1-1003.

1-7. Life Cycle Project Management Applicability

Project control may be used through the entire life cycle of a project, spanning decades in many cases. During initial reconnaissance surveys of a project, control should be permanently monumented and situated in areas that are conducive to the performance or densification of subsequent surveys for contract plans and specifications, 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 duplicate or redundant surveys to the maximum extent possible.

1-8. Metrics

Both non-SI and metric units are used in this manual. Metric units are commonly used in control surveying applications, including the control survey work covered in this manual. Control surveyed geographical or metric Cartesian coordinates are generally transformed to non-SI 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 either metric or non-SI units shall follow local engineering and construction practices. Non-SI/metric equivalences are noted where applicable, including the critical--and often statutory--distinction between the U.S. Survey Foot (1,200/3,937 meters (m) exactly) and International Foot (30.48/100 m exactly) conversions.

1-9. Trade Name Exclusions

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

1-10. Accompanying Guide Specifications

Appendix C provides guide specifications which can be used to support A-E service contracts for control surveying.

1-11. CORPSCON

A conversion program, CORPSCON, that performs datum conversions for the continental U.S. has been developed by the U.S. Army Topographic Engineering Center (TEC). Technical documentation and operating instructions are given in Appendix D.

Chapter 2
Control Surveying Applications

2-1. General

Control surveys are used to support project control densification, structural deformation studies, photogrammetry, dynamic positioning and navigation for hydrographic survey vessels and dredges, hydraulic study/survey location, river/floodplain cross-section location, core drilling location, environmental studies, levee overbank surveys, levee profiling, levee grading and revetment placement, disposal area construction, grade control, support for real estate surveys and regulatory enforcement actions.

2-2. Project Control Densification

a. Conventional surveying. Conventional control surveys can be used to economically and accurately establish or densify project control in a timely fashion. Quality control statistics and redundant measurements in networks established by these methods help to ensure reliable results. However, conventional survey methods do have the requirement for intervisibility between adjacent stations.

b. GPS surveying. GPS survey techniques can often be used to establish or densify project control more efficiently than conventional control surveying techniques. Quality control statistics and redundant measurements in GPS networks help to ensure reliable results. Field operations to perform a GPS survey are relatively easy and can generally be performed by one person per receiver, with two or more receivers required to transfer control. GPS does not require intervisibility between adjacent stations. However, GPS must have visibility of at least four satellites (for position determination) during surveying. This requirement may make GPS inappropriate in areas of dense vegetation. For GPS control survey techniques refer to EM 1110-1-1003.

2-3. Geodetic Control Densification

Conventional control and GPS surveying methods can be used for wide-area high-order geodetic control densification. First-, Second- or Third-order work can be achieved using conventional or GPS surveying techniques.

2-4. Vertical Control Densification

Orthometric heights (from benchmarks located in the project area) and conventional leveling methods are used to determine elevations (orthometric heights) for vertical control densification. The setup and operation for conventional control surveying for vertical control densification offers economies of scale in the same manner as that offered by the setup for project control densification: smaller projects require less setups, while larger projects require more. The procedures for using GPS for vertical control densification are covered in further detail in EM 1110-1-1003.

2-5. Structural Deformation Studies

a. Conventional control surveying can be used to monitor the motion of points on a structure relative to stable monuments. This can be done with an array of calibrated reflectors positioned at selected points on the structure, an Electronic Distance Measuring instrument (EDM) alternated on various remote stable monuments, and trilateration techniques. These precise techniques can provide a direct measure of the displacement of a structure as a function of time. If procedures are strictly adhered to, it is possible to achieve $\pm$0.5 mm + 4 ppm (4 mm/km) for baseline using conventional surveying. Personnel requirements generally are two personnel once the initial test network of reference and object points are set up: one person to monitor the EDM, another to aid in reflector placement.

b. GPS can be used to monitor the motion of points on a structure relative to stable monuments. With GPS, an array of antennae are positioned at selected points on the structure and on remote stable monuments as opposed to using reflectors and EDMs as previously described. The baselines between the antennae are formulated to monitor differential movement. The relative precision of the measurements is on the order of $\pm$5 mm over distances averaging between 5 and 10 km. Formulations can be determined continuously 24 hours a day, depending on GPS constellation availability. Once a deformation monitoring system has been set up using GPS, it can be operated unattended and is relatively easy to maintain. More specific guidance on the use of GPS for deformation monitoring is included in EM 1110-1-1003.

c. Methods for deformation determination, including the direct measurement of deformation parameters (e.g., tilt, strain, stress, etc.), the real-time processing of continuously recorded deformation data, the structural finite element method, and the integrated analysis of deformation measurements, are not covered in this manual. For further information on these techniques, the user should consult any of the references listed in Appendix A relative to the particular subject.

2-6. Photogrammetry

Conventional control and GPS surveying can be used in the support of photogrammetric applications. More specific guidance on the use of control surveying in support of photogrammetry is included in EM 1110-1-1000.

2-7. Dynamic Positioning and Navigation

a. Conventional control surveying can be used to establish control for the dynamic positioning and navigation of construction and surveying platforms used for design, construction, and environmental regulatory efforts. These efforts include dredge control systems, site investigation studies/surveys, horizontal and vertical construction placement, hydraulic studies, or any other activity requiring two- or three-dimensional control. Second- or Third-Order leveling is required for these efforts.

b. GPS can reduce the time and effort required to install control for dynamic positioning and navigation. In addition to this capability, properly equipped GPS equipment can provide dynamic, real-time GPS code and carrier phase positioning of construction and surveying platforms. GPS code phase differential techniques can provide real-time meter-level horizontal positioning and navigation, while GPS carrier phase differential techniques can provide real-time centimeter-level three-dimensional positioning and navigation. These GPS methods can be used for any type of construction or survey platform (e.g., dredges, graders, survey vessels, etc.). More specific guidance on the use of GPS for dynamic positioning and navigation is included in EM 1110-1-1003.

2-8. GIS Integration

A Geographic Information System (GIS) can be used to correlate and store diverse information on natural or man-made characteristics of geographic positions. To effectively establish and use a GIS, it must be based on accurate geographic coordinates. A GIS with an accurate foundation of geographic coordinates enables the user to readily exchange information between databases. Conventional control surveying and GPS surveying can be used to establish the geographic coordinates used as the foundation for a GIS.

Chapter 3
Standards and Specifications for Control Surveying Applications

3-1. General

This chapter details standards and specifications for control surveying applications and provides guidance on how to reach them.

3-2. Accuracy

a. This section prescribes field surveying standards and specifications that are applicable to surveys performed to provide base reference control, regardless of the application, whether in support of the engineering design, planning, construction, project operation and maintenance, real estate, and/or regulatory enforcement functions.

NOTE: The accuracy of control surveying measurements should be consistent with the purpose of the survey. When evaluating the technique to be used and accuracies desired, the surveyor must evaluate the limits of the errors of the equipment involved, the procedures to be followed, and the error propagation. These evaluations should be firmly based on past experience or written guidance. It is important to remember in this evaluation that the best survey is the one that provides the data at the required accuracy levels without wasting manpower, time, and money.

b. Survey accuracy standards prescribed in this section relate to the relative accuracy derived from a particular survey. This relative accuracy (or precision) is estimated by internal closure checks of the survey run through the local project, map, or construction site. Relative survey accuracy estimates are traditionally expressed as ratios of the misclosure to the total length of the survey (e.g., 1:10,000). Relative survey accuracies are different than map accuracies, which are expressed in terms of limiting positional error. Since map compilation is dependent on survey control, map accuracies will ultimately hinge on the adequacy and accuracy of the base survey used to control the map.

c. Tables 3-1 and 3-2 detail the basic minimum criteria required for planning, performing, and evaluating the adequacy of control surveys.

d. These criteria apply to all conventional control surveying and GPS surveying activities regardless of the intended accuracies.

e. Many of the criteria shown in the tables are developed from FGCS standards for performing conventional control surveys and GPS surveys. The criteria listed in the tables have been modified to provide more practical standards for engineering and survey densification. FGCS Standards and Specifications for Geodetic Control Networks covers all aspects of performing conventional control surveys for high-precision geodetic network densification purposes, while FGCS GPS Standards covers the use of GPS surveys for the same application. This intended application of techniques is not practical for typical civil works, military construction, and environmental restoration activities where lower precision control is acceptable.

f. If a primary function of a survey is to support NGRS densification, then specifications listed in the FGCS publications (1984 and 1988) should be followed in lieu of those in Tables 3-1 and 3-2.

(1) Survey classification. A survey shall be classified based on its horizontal point closure ratio, as indicated in Table 3-1 or the vertical elevation difference closure standard given in Table 3-2.

Table 3-1
USACE Point Closure Standards for Horizontal Control Surveys

USACE Classification	Point Closure Standard (Ratio)
Second Order Class I	1:50,000
Second Order Class II	1:20,000
Third Order Class I	1:10,000
Third Order Class II	1: 5,000
Construction Layout/ Fourth Order	1:2,500 - 1:20,000

Table 3-2
USACE Point Closure Standards for Vertical Control Surveys

USACE Classiciation	Point Closure Standard (Feet)
Second Order Class I	$0.025*M^{0.5}$
Second Order Class II	$0.035*M^{0.5}$
Third Order	$0.050*M^{0.5}$
Construction Layout/Fourth Order	$0.100*M^{0.5}$

NOTE: $M^{0.5}$ = square root of distance M in miles

(2) Horizontal control standards. The horizontal point closure is determined by dividing the linear distance misclosure of the survey into the overall circuit length of a traverse, loop, or network line/circuit. When independent directions or angles are observed, as on a conventional survey (i.e., traverse, trilateration, or triangulation), these angular misclosures may optionally be distributed before assessing positional misclosure. In cases where GPS vectors are measured in geocentric coordinates, then the three-dimensional positional misclosure is assessed.

(a) Approximate surveying. Approximate surveying work should be classified based on the survey's estimated or observed positional errors. This would include absolute GPS and some differential GPS techniques with positional accuracies ranging from 10 to 150 feet (2DRMS). There is no order classification for such approximate work.

(b) Higher order survey. Requirements for relative line accuracies exceeding 1:50,000 are rare for most USACE applications. Surveys requiring accuracies of First-Order (1:100,000) or better should be performed using FGCS standards and specifications, and must be adjusted by the NGS.

(c) Construction layout or grade control. The Construction Layout or Grade Control USACE Classification is analogous to traditional Fourth-Order work. This classification is intended to cover temporary control used for alignment, grading, and measurement of various types of construction, and some local site plan topographic mapping or photo mapping control work. Accuracy standards will vary with the type of construction. Lower accuracies (1:2,500 - 1:5,000) are acceptable for earthwork, embankment, beach fill, and levee alignment stakeout and grading, and some site plan, curb and gutter, utility building foundation, sidewalk, and small roadway stakeout. Moderate accuracies (1:5,000) are used in most pipeline, sewer, culvert, catch basin, and manhole stakeout, and for general residential building foundation and footing construction, major highway pavement, and concrete runway stakeout work. Somewhat higher accuracies (1:10,000 - 1:20,000) are used for aligning longer bridge spans, tunnels, and large commercial structures. For extensive bridge or tunnel projects, 1:50,000 or even 1:100,000 relative accuracy alignment work may be required. Vertical grade is usually observed to the nearest 0.01 foot for most construction work, although 0.1-foot accuracy is sufficient for riprap placement, grading, and small diameter pipe placement. Construction control points are typically marked by semi-permanent or temporary monuments (e.g., plastic hubs, P-K nails, wooden grade stakes). Control may be established by short, non-redundant spur shots, using total stations or GPS, or by single traverse runs between two existing permanent control points. Positional accuracy will be commensurate with, and relative to, that of the existing point(s) from which the new point is established.

(3) Vertical control standards. The vertical accuracy of a survey is determined by the elevation misclosure within a level section or level loop. For conventional differential or trigonometric leveling, section or loop misclosures (in feet) shall not exceed the limits shown in Table 3-2, where the line or circuit length (M) is measured in miles. Fourth-Order accuracies are intended for construction layout grading work. Procedural specifications or restrictions pertaining to vertical control surveying methods or equipment should not be overly restrictive.

(4) Contract compliance with USACE survey standards. Contract compliance assessment shall be based on the prescribed point closure standards of internal loops, not on closure with external networks of unknown accuracy. In cases where internal loops are not observed, then assessment must be based on external closures. Specified closure accuracy standards shall not be specified that exceed those required for the project, regardless of the accuracy capabilities of the survey equipment.

3-3. General Procedural Standards and Specifications

a. Most survey applications in typical civil works and military arenas can be satisfied with a Second- or Third-Order level of accuracy. Higher levels of accuracy are required for the densification of high precision geodetic networks and some forms of deformation monitoring.

b. Since most modern survey equipment (e.g., GPS or electronic total stations) are capable of achieving far higher accuracies than those required for engineering, construction, and mapping, only generalized field survey specifications are necessary for most USACE work. The following paragraphs outline some of the more critical specifications which relate to the USACE horizontal and vertical standards. Additional guidance for performing control surveying is found in subsequent chapters, as well as many of the technical manuals listed in Appendix A.

(1) Survey instrumentation criteria. USACE Commands shall minimize the use of rigid requirements for particular surveying equipment or instruments used by professional surveying contractors. In some instances,

contract technical specifications may prescribe a general type of instrument system be employed (e.g., total station, GPS, spirit level), along with any unique operating or calibration requirements.

(2) Survey geometry and field observing criteria. In lieu of providing detailed government procedural specifications, professional contractors may be presumed capable of performing surveys in accordance with accepted industry standards and practices. Any geometrical form of survey network may be formed: traverses, loops, networks, and cross-links within networks. Traverses should generally be closed back (or looped) to the same point, to allow an assessment of the internal misclosure accuracies. Survey alignment, orientation, and observing criteria should not be rigidly specified; however, guidance regarding limits on numbers of traverse stations, minimum traverse course lengths, auxiliary connections, etc. are provided in the subsequent chapters of this EM, as well in other EM's listed in Appendix A.

(3) Connections to existing control. USACE surveys should be connected to existing local control or project control monuments/benchmarks. These existing points may be those of any Federal (including USACE project control), state, local, or private agency. Ties to local USACE project control and boundary monuments are absolutely essential and critical to design, construction, and real estate. In order to minimize scale or orientation errors, at least two existing monuments should be connected, if practicable. However, survey quality control accuracy assessments (Tables 3-1 and 3-2) shall be based on internal traverse or level line closures--not on external closures between or with existing monuments or benchmarks. Accuracy assessments based on external closures typically require a knowledge of the statistical variances in the fixed network.

(4) Connections to NGRS control. The NGRS pertains to geodetic control monuments with coordinate or elevation data published by the NGS. It is recommended that USACE surveys be connected with one or more stations on the NGRS when practicable and feasible. Connections with the NGRS shall be subordinate to the requirements for connections with local/project control. Connections with local/project control that has previously been connected to the NGRS are adequate in most cases.

(5) Survey datums. A variety of survey datums and references are used throughout USACE projects. It is recommended that horizontal surveys be referenced to either the North American Datum of 1983 (NAD 83) or NAD 27 systems, with coordinates referred to the local SPCS for the area. The Universal Transverse Mercator (UTM) grid system may be used for military operational or tactical uses, in OCONUS locales without a local coordinate system, or on some civil projects crossing multiple SPCS zones. Vertical control should be referenced to either the National Geodetic Vertical Datum, 1929 Adjustment (NGVD 29) or the North American Vertical Datum, 1988 Adjustment (NAVD 88). Independent survey datums and reference systems shall be avoided unless required by local code, statute, or practice. This includes local tangent grid systems, state High Accuracy Reference Networks (HARN), and unreferenced construction baseline station-offset control.

(6) Spur points. Spur points (open-ended traverses or level lines) should be avoided to the maximum extent practicable. In many cases, it is acceptable survey procedure for temporary Fourth-Order construction control, provided adequate blunder detection is taken. Kinematic differential GPS (DGPS code or carrier phase tracking) surveys are effectively spur point surveys with relative accuracies well in excess of Third-Order standards. These DGPS kinematic spur techniques may be acceptable procedures for most control surveying in the future, provided blunder protection procedures are developed. Refer to EM 1110-1-1003 for further GPS guidance.

(7) Survey adjustments. The standard adjustment method in USACE will be either the Compass Rule or Least Squares. Technical (and contractual) compliance with accuracy standards prescribed in Tables 3-1 and 3-2 will be based on the internal point misclosures. Propagated relative distance/line accuracy statistics used by FGCS that result from unconstrained (minimally constrained) least squares adjustment error propagation statistics may be assumed comparable to relative misclosure accuracy estimates for survey quality control assessment. Surveys may be adjusted (i.e., constrained) to existing control without regard to the variances in the existing network adjustment. Exceptions to this requirement are surveys performed to FGCS standards and specifications that are adjusted by NGS.

(8) Data recording and archiving. Field survey data may be recorded either manually or electronically. Manual recordation should follow industry practice, using formats outlined in various technical manuals (Appendix A). Refer to EM 1110-1-1005 for electronic survey data collection standards.

3-4. Construction Surveys

In-house and contracted construction surveys generally will be performed to meet Third-Order-Class II (1:5,000) accuracy. Some stake-out work for earthwork clearing and grading, and other purposes, may need only be performed to meet Fourth-Order accuracy requirements. Other stake-out work, such as tunnel or bridge pier alignment, may require Second-Order or higher accuracy criteria. Construction survey procedural specifications should follow recognized industry practices. Reference also FM 5-233, Construction Surveying.

3-5. Cadastral and Real Estate Surveys

Many state codes, rules, or statutes prescribe minimum technical standards for surveying and mapping. Generally, most state accuracy standards for real property surveys parallel USACE Third-Order point closure standards--usually ranging between 1:5,000 and 1:10,000. USACE and its contractors shall follow applicable state minimum technical standards for real property surveys involving the determination of the perimeters of a parcel or tract of land by establishing or reestablishing corners, monuments, and boundary lines, for the purpose of describing, locating fixed improvements, or platting or dividing parcels. Although state minimum standards relate primarily to accuracies of land and boundary surveys, other types of survey work may also be covered in some areas. See also ER 405-1-12, Real Estate Handbook. Reference also the standards and specifications prescribed in the "Manual of Instruction for the Survey of the Public Lands of the United States" (U.S. Bureau of Land Management 1947) for cadastral surveys, or surveys of private lands abutting or adjoining Government lands.

3-6. Geodetic Control Surveys

a. Geodetic control surveys are usually performed for the purpose of establishing a basic framework to be included in the national geodetic reference network, or NGRS. These geodetic survey functions are distinct from the survey procedures and standards defined in this EM which are intended to support USACE engineering, construction, mapping, and Geographic Information System (GIS) activities.

b. Geodetic control surveys of permanently monumented control points that are to be incorporated in the NGRS must be performed to far more rigorous standards and specifications than are surveys used for general engineering, construction, mapping, or cadastral purposes. When a project requires NGRS densification, or such densification is a desirable by-product and is economically justified, USACE Commands should conform to FGCS survey standards and specifications, and other criteria prescribed under Office of Management and Budget (OMB) Circular A-16 (OMB 1990). This includes related automated data recording, submittal, project review, and adjustment requirements mandated by FGCS and NGS. Details outlining the proposed use of FGCS standards and specifications in lieu of USACE standards, including specific requirements for connections to the NGRS, shall be included in the descriptions of survey and mapping activities contained in project authorization documents.

c. Geodetic control surveys intended for support to and inclusion in the NGRS must be done in accordance with the following FGCS publications:

(1) "Standards and Specifications for Geodetic Control Networks" (FGCS 1984).

(2) "Geometric Geodetic Accuracy Standards and Specifications for Using GPS Relative Positioning Techniques" (FGCS 1988).

(3) "Input Formats and Specifications of the National Geodetic Data Base" (also termed the "Bluebook") (FGCS 1980).

(4) "Guidelines for Submitting GPS Relative Positioning Data to the National Geodetic Survey" (NGS 1988).

A survey performed to FGCS accuracy standards and specifications cannot be definitively classified or certified until the NGS has performed a variance analysis of the survey relative to the existing NGRS. This analysis and certification cannot be performed by USACE Commands--only the NGS can perform this function. It is estimated that performing surveys to meet these FGCS standards, specifications, and archiving criteria can add between 25 and 50 percent to the surveying costs of a project. Therefore, sound judgment must be exercised on each project when determining the practicability of doing survey work that, in addition to meeting the needs of the project, can be used for support to and inclusion in the NGRS.

3-7. Topographic Site Plan Mapping Surveys

Control surveys from which site plan mapping is densified (using plane tables, electronic total stations, or GPS) are normally established to USACE Third-Order standards. Follow guidance in FM 5-233, Construction Surveying;

FM 5-82D, Topographic Surveyor; FM 5-232, Geodetic and Topographic Surveying; EM 1110-1-1005.

3-8. Structural Deformation Surveys

Structural deformation surveys are performed in compliance with the requirements in ER 1110-2-100, often termed PICES surveys. PICES surveys require high line vector and/or positional accuracies to monitor the relative movement of monoliths, walls, embankments, etc. PICES survey accuracy standards vary with the type of construction, structural stability, failure probability, and impact, etc. In general, horizontal and vertical deformation monitoring survey procedures are performed relative to a control network established for the structure. Ties to the NGRS or NGVD 29 are not necessary other than for general reference; and then only an USACE Third-Order connection is needed. FGCS geodetic relative accuracy standards are not applicable to these localized movement surveys. Other deformation survey and instrumentation specifications and procedures for earth and rock fill dams and concrete structures are in EM 1110-2-1908 and EM 1110-2-4300.

3-9. Photogrammetric Mapping Control Surveys

Control surveys required for controlling photogrammetric mapping products will normally be performed to USACE Third-Order standards. Occasionally USACE Second-Order standards will be required for extensive aerotriangulation work. Reference EM 1110-1-1000, for detailed photogrammetric mapping standards and specifications.

3-10. Hydrographic Surveys

Control points for USACE hydrographic surveys generally are set to Third-Order horizontal and vertical accuracy. Exceptions are noted in EM 1110-2-1003. Hydrographic depth sounding accuracies are based on the linear and radial error measures described in EM 1110-2-1003, as well as hydrographic survey procedural specifications. Requirements for compliance with EM 1110-2-1003 are contained in ER 1130-2-307.

3-11. GIS Surveys

GIS raster or vector features can be scaled or digitized from any existing map. Typically a standard USGS 1:24,000 quadrangle map is adequate given the accuracies needed between GIS data features, elements, or classifications. Relative or absolute GPS (i.e., 10- to 30-foot precision) survey techniques may be adequate to tie GIS features where no maps exist. Second- or Third-Order control networks are generally adequate for all subsequent engineering, construction, real estate, GIS, and/or Automated Mapping/Facilities Management (AM/FM) control.

Chapter 4
Reference Systems and Transformations

4-1. General

The discipline of surveying consists of locating points of interest on the surface of the earth. Points of interest are defined by spherical or planar coordinate values that are referenced to a defined mathematical figure. In surveying, this mathematical figure may be an equipotential surface, ellipsoid of revolution, or a plane.

a. Geoid. The geoid is an equipotential surface where the plumb line is perpendicular to each point on its surface. The geoid is considered a mean sea level (MSL) surface extended continuously through the continents. The geoidal surface is irregular due to mass excesses and deficiencies within the earth (Figure 4-1). The geoidal surface is the reference system for orthometric heights and astronomic coordinates.

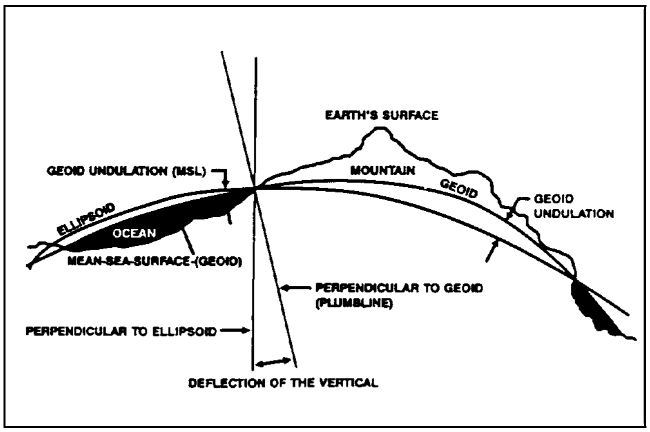


Figure 4-1. The relationship between the ellipsoid, geoid, and the physical surface of the Earth

b. Ellipsoid. An ellipsoid of revolution is a mathematical figure that "best" approximates the geoid (Figure 4-1). The ellipsoid of revolution is developed by rotating an ellipse about its semi-minor axis (b) and is symmetric to the equator.

Ellipsoid of revolution:

$$X^2/a^2 + Y^2/a^2 + Z^2/b^2 = 1$$

where

X,Y,Z = Cartesian coordinates

a = semi-major axis

b = semi-minor axis

The ellipsoid of revolution provides a defined mathematical surface to calculate geodetic distances, azimuths, and coordinates. Some adjustment, transformation, and GPS postprocessing software packages require the user to input the size and shape of the reference ellipsoid. The semi-major (a) and semi-minor (b) axes are used to determine the ellipsoid size (Figure 4-2). The ellipsoid shape can also be defined by the flattening (f), reciprocal of flattening (f^{-1}), eccentricity, and second eccentricity (e').

Flattening: $f = (a-b)/a$

Reciprocal of flattening: $f^{-1} = 1/f$

Eccentricity: $e = (a^2 - b^2)^{0.5}/a$ *or*
$e = (2f - f^2)^{0.5}$

Second eccentricity: $e' = (a^2 - b^2)^{0.5}/b$

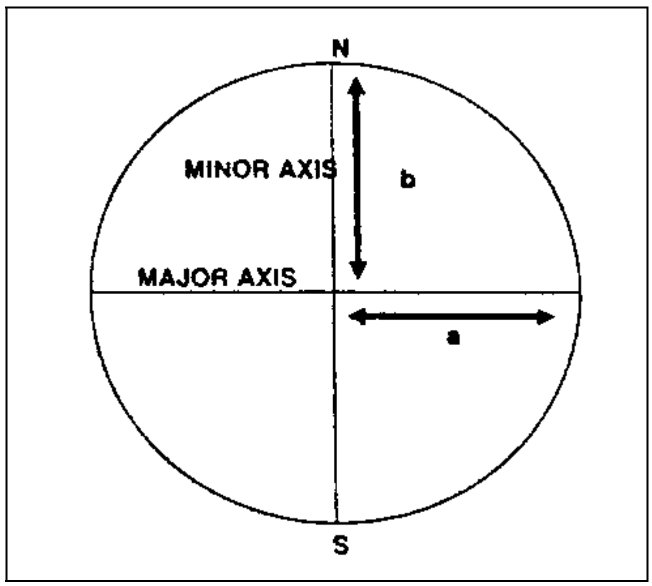


Figure 4-2. Ellipse

Table 4-1 contains the ellipsoid parameters for the Clarke 1866, Geodetic Reference System 1980 (GRS 80), and the World Geodetic System 1984 (WGS 84) reference ellipsoids. The Clarke 1866 and GRS 80 are the mathematical surfaces utilized by the North American Datum 1927

Table 4-1
Ellipsoidal Parameters

Datum	
NAD 27	
Ellipsoid	Clark 1866
Semi-major axis (a)	6,378,206.4 m
Flattening (1/f)	294.9786982
NAD 83	
Ellipsoid	GRS 80
Semi-major axis (a)	6,378,137.0 m
Flattening (1/f)	298.257222101
WGS 84	
Ellipsoid	WGS 84
Semi-major axis (a)	6,378,137.0 m
Flattening (1/f)	298.257223563

(NAD 27) and the North American Datum 1983 (NAD 83). The WGS 84 is the reference system utilized by the GPS. The WGS 84 and NAD 83 are considered synonymous in the continental United States. However, in some locations, positional variations of several meters may occur between the NAD 83 and WGS 84.

c. Cartesian coordinates (X, Y, Z). Cartesian coordinates are considered a true three-dimensional coordinate system. They can be referenced to a regional ellipsoid (Clarke 1866), a global ellipsoid (WGS 84), a plane, or a single point. Cartesian coordinates are termed geocentric if they are related to a global ellipsoid which has its coordinate origin at the mass center of the earth. However, Cartesian coordinates are seldom utilized in engineering and cadastral surveys.

d. Geodetic coordinates. Geodetic coordinates consist of latitude (ϕ), longitude (λ), and ellipsoid height (h). Geodetic latitude, longitude, and ellipsoid height define the position on the surface of the earth with respect to the reference ellipsoid (Figure 4-3).

(1) Geodetic latitude (ϕ). The geodetic latitude of a point is the angle between the equatorial plane and the normal through the point on the ellipsoid (Figure 4-3). Geodetic latitude is positive north of the equator and negative south of the equator.

(2) Geodetic longitude (λ). The geodetic longitude is the angle measured in the equatorial plane from the prime meridian (Greenwich meridian) to the defined point (Figure 4-3). Longitude is commonly measured eastward from the Greenwich meridian (0°-360°). In the continental United States, longitudes are expressed westerly. To convert easterly to westerly referenced longitudes, the easterly longitude must be subtracted from three hundred and sixty degrees (360°).

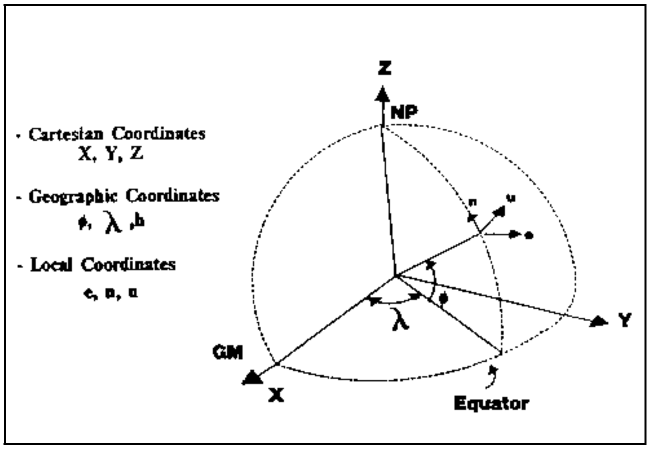


Figure 4-3. Coordinate reference frames

East-west longitude conversion:

λ = 282 52 36.345 E

λ = 360 - 282 52 36.345 E

λ = 77 07 23.655 W

(3) Ellipsoid height (h). The ellipsoid height above the reference ellipsoid is the distance measured along the ellipsoidal normal to the point in question. The ellipsoid height is positive if the reference ellipsoid is below the topographic surface and negative if the ellipsoid is above the topographic surface. The ellipsoid height is not utilized in engineering and cadastral surveys.

(4) Geoid separation (N). The geoid separation is the distance between the normal to the reference ellipsoid and the geoid. The geoid separation is positive if the geoid is above the ellipsoid and negative if the geoid is below the ellipsoid.

(5) Orthometric height (H). The orthometric height is the vertical distance of a point above or below the geoid. In the United States, elevations are commonly referenced to MSL which approximates the geoidal surface. Orthometric (MSL) heights are utilized in engineering, construction, and topographic surveys.

e. Datum. A datum is a numerical or geometrical reference system. Horizontal and vertical datums are commonly used in surveying and mapping. Horizontal and vertical datums are further subdivided as project and geodetic.

(1) Horizontal datum. A horizontal datum is defined by the geometric figure utilized (plane, ellipsoid, sphere) in coordinate, distance, and directional calculations; initial reference point (origin); and a defined azimuth or bearing from the initial point.

(a) Geodetic datum. Five parameters are required to define a geodetic datum: the semi-major axis (a) and flattening (f) define the size and shape of the reference ellipsoid; the latitude and longitude of an initial point; and a defined azimuth (α) from the initial point. The NAD 27 and NAD 83 are examples of geodetic datums. For example, the NAD 27 utilizes the Clarke 1866 ellipsoid. The initial point for the NAD 27 is at Meades Ranch, Kansas, and a defined azimuth extends from Meades Ranch to Station Waldo.

(b) Project. A project datum is relative to local control and may not be directly referenced to a geodetic datum. Project datums are the most common reference system utilized by the Corps of Engineers.

(c) The adjusted horizontal datums typically used are the NAD 27 and NAD 83. The FGCS, of which USACE is a member, has adopted the NAD 83 as the horizontal datum for surveying and mapping activities performed or financed by the Federal Government. To the extent practicable, legally allowable, and feasible, the USACE should use NAD 83 in its surveying and mapping activities. Transformations between NAD 27 and NAD 83 are done using the *CORPS Con*vert (i.e., CORPSCON) software package or other *N*orth *A*merican *D*atum *C*onversion (i.e., NADCON) based program whenever a survey or resurvey with new control or a readjustment of the original survey observations cannot be done. (See Appendix D.)

(d) The NAD 27 is based on an adjustment of the Clarke 1866 reference ellipsoid. The origin and orientation of NAD 27 is defined relative to a fixed triangulation station in Kansas (i.e., Meades Ranch). Azimuth orientation for NAD 27 is South. The original network adjustment for NAD 27 included approximately 25,000 stations. The best fitting area for the resultant adjustment is North America. The longitude origin for NAD 27 is the Greenwich Meridian. The reference units are U.S. Survey Feet and the coordinate values are defined in terms of x and y.

(e) The NAD 83 is defined relative to the Geodetic Reference System of 1980 (GRS 80). GRS 80 is an ellipsoid model based on the mass center of the earth. Azimuth orientation for NAD 83 is North. The original network adjustment for NAD 83 included approximately 250,000 stations. The best fitting area for the resultant adjustment is worldwide. The longitude origin for NAD 83 is the Greenwich Meridian. The reference units are meters and the coordinate values are defined in terms of easting and northing.

(2) Vertical datum.

(a) A vertical datum is a reference system for elevations. Vertical datums are most commonly referenced to MSL, mean low water (MLW), mean low low water (MLLW), or mean high water (MHW). MSL elevations are utilized for construction, photogrammetric, geodetic, and topographic surveys. MLW elevations are utilized in dredging projects. MHW elevations are utilized in construction projects involving bridges and tunnels.

(b) The vertical reference system most often used in the past was the NGVD 29. The NAVD 88 is to be used for all future work. Transformations between NGVD 29 and NAVD 88 will be done using the *Vert*ical *Con*version (i.e., VERTCON) software whenever a survey or resurvey with new control or a readjustment of the original survey observations cannot be done.

4-2. The SPCS

The SPCS was developed by the NGS to provide a planar representation of the earth's surface. To properly relate spherical coordinates (ϕ,λ) to a planar system (northings and eastings), a developable surface must be constructed. A developable surface is defined as a surface that can be expanded without stretching or tearing. The two most common developable surfaces or map projections used in surveying and mapping are the cone and cylinder (Figure 4-4). The projection of choice is dependent on the north-south or east-west areal extent of the region. Areas with limited east-west dimensions and indefinite north-south extent utilize the Transverse Mercator (TM) projection. Areas with limited north-south dimensions and indefinite east-west extent utilize the Lambert projection. The SPCS was designed to minimize the distortion at a point to approximately one-part in ten thousand (1:10,000). To achieve this criterion, the SPCS has been divided into zones that have a maximum width or height of approximately one-hundred and fifty eight statute miles (158 miles). To minimize distortion, each state may have several zones or may employ both the Lambert and TM projections. For example, Florida consists of one Lambert zone that encompasses the panhandle region and two TM zones that cover the remainder of the state. The TM and Lambert projections are conformal. A conformal

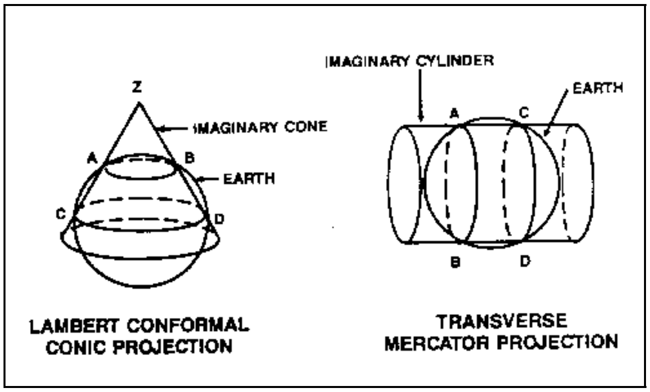


Figure 4-4. Common map projections

projection preserves angular relationships. Angles measured on the ellipsoid are equal to angles measured on a conformal projection. To calculate TM or Lambert projection state plane coordinates, the user must identify the datum to which the geodetic coordinates are referenced.

a. The TM. The TM projection utilizes a cylindrical surface. The cylinder is perpendicular to the rotation axis of the ellipsoid and intersects the ellipsoid along two ellipses equidistant from the central meridian (Figure 4-5). Distortions in the TM projection increase in the east-west direction; therefore, the TM projection is utilized in states with north-south extent. The TM scale factor is unity where the cylinder intersects the ellipsoid. The scale factor is less than one within the lines of intersection and greater than one outside the lines of intersection (Figure 4-5). The scale factor is the ratio of arc length on the projection to arc length on the ellipsoid. To compute the state plane coordinates of a point, the latitude and longitude of the point and the projection constants for that zone or state must be known.

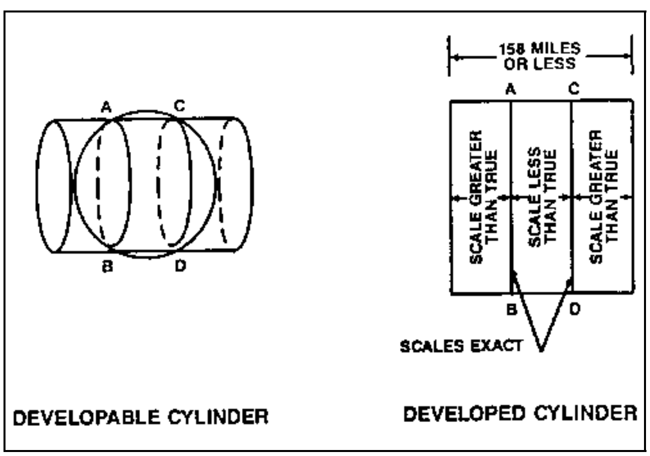


Figure 4-5. TM projection

b. Lambert conformal conic. The Lambert projection utilizes a cone that is coincident with the rotational axis of the ellipsoid and intersects the ellipsoid along two standard parallels (Figure 4-6). The scale factor is equal to unity at the standard parallels and is less than one inside and greater than one outside the standard parallels. The scale factor remains constant along the parallel; therefore, the Lambert projection is ideal for states with indefinite east-west extent.

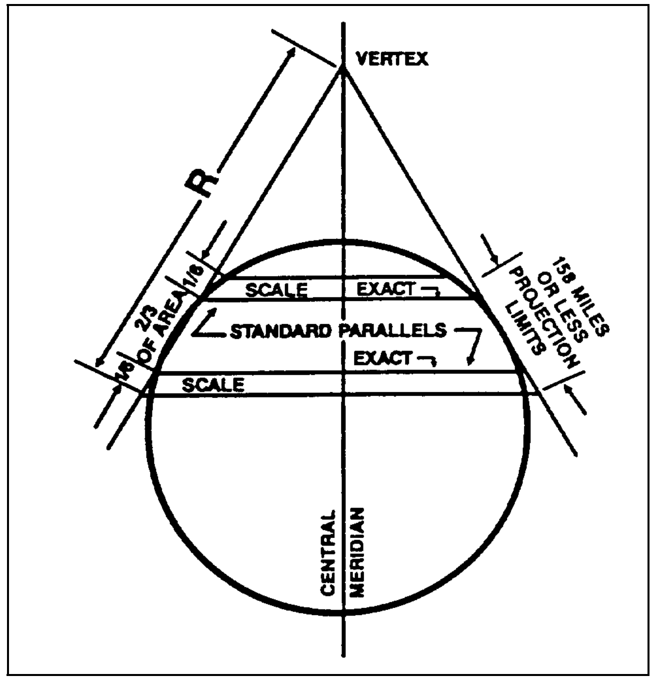


Figure 4-6. Lambert projection

c. Grid distances. Grid distances are computed by multiplying the horizontal distance measured in the field by the combination or grid factor. The combination factor consists of the sea level and scale factor. The sea level factor is computed by dividing the mean radius of the earth by the sum of the mean earth radius and the average elevation between the points. The mean radius of the earth is approximately twenty million nine-hundred and six thousand feet (20,906,000 ft or 68,589,101.667 m). The sea level distance is obtained by multiplying the sea level factor by the measured distance.

Sea level factor:

$$SL = R/(R + H)$$

where

SL = sea level factor

R = mean earth radius

H = average elevation

Example

Measured distance = 2,623.000 feet

Average elevation (H) = 300 feet

Sea level (SL) factor = 0.999985650259

Reduced distance = 2,622.962 feet

The scale factor is a function of latitude and can be interpolated from tables published from the National Oceanic and Atmospheric Administration (NOAA).

d. Units. State plane coordinates can be expressed in both feet and meters. State plane coordinates defined on the NAD 27 are published in feet. State plane coordinates defined on the NAD 83 are published in meters; however, state and Federal agencies can request the NGS to provide coordinates in feet. If NAD 83 state plane coordinates are defined in meters and the user intends to convert those values to feet, the proper meter-foot conversion factor must be utilized. Some states utilize the International Foot rather than the U.S. Survey Foot in the conversion of feet to meters.

International Foot:

1 International Foot = 0.3048 meter (Exact)

U.S. Survey Foot:

1 U.S. Survey Foot = 1200/3937 meter (Exact)

e. NAD 83 versus NAD 27 state plane coordinates. The major difference between NAD 83 and NAD 27 state plane coordinates is that NAD 83 state plane coordinates are published in meters. Also, in the establishment of the NAD 83 SPCS, some zones that were present in the NAD 27 system were eliminated. Future USACE projects should adopt the NAD 83 SPCS.

4-3. Universal Transverse Mercator (UTM)

UTM coordinates are used in surveying and mapping when the size of the project extends through several state plane zones or projections. UTM coordinates are also utilized by the U.S. Army, Air Force, and Navy for mapping, charting, and geodetic applications. The UTM projection differs from the TM projection in the scale at the central meridian, origin, and unit representation. The scale at the central meridian of the UTM projection is 0.9996. In the northern hemisphere, the northing coordinate has an origin of zero at the equator. In the southern hemisphere, the southing coordinate has an origin of ten million meters (10,000,000 m). The easting coordinate has an origin of five-hundred thousand meters (500,000 m) at the central meridian. The UTM system is divided into sixty (60) longitudinal zones. Each zone is six (6) degrees in width extending three (3) degrees on each side of the central meridian. The UTM system is applicable between latitudes eighty-four degrees north (84 N) to eighty degrees south (80 S). To compute the UTM coordinates of a point, the TM coordinates must be determined. The UTM northing or southing (N_{UTM}, S_{UTM}) coordinates are computed by multiplying the scale factor (0.9996) at the central meridian by the TM northing or southing (N_{TM}, S_{TM}) coordinate values. In the southern hemisphere, a ten-million-meter (10,000,000-m) offset must be added to account for the origin. The UTM eastings (E_{UTM}) are derived by multiplying the TM eastings (E_{TM}) by the scale factor of the central meridian (0.9996) and adding a five-hundred-thousand-meter (500,000-m) offset to account for the origin. UTM coordinates are always expressed in meters.

UTM northings, southings, and eastings

$$N_{UTM} = (0.9996)N_{TM} \text{ (Northern Hemisphere)}$$

$$S_{UTM} = (0.9996)S_{TM} + 10{,}000{,}000 \text{ m (Southern Hemisphere)}$$

$$E_{UTM} = (0.9996)E_{TM} + 500{,}000 \text{ m}$$

The UTM zone (Z) can be calculated by knowing the geodetic longitude of the point. In the continental United States the UTM zones range from ten (10) to nineteen (19). If the value **Z** is a decimal quantity, the zone must be incremented by one. In the example below **Z** is a decimal quantity; therefore, the zone equals seventeen (17) plus one (1).

Zone

$$Z = (180 + \lambda)/6 \text{ (Easterly Longitude)}$$

$$Z = (180 - \lambda)/6 \text{ (Westerly Longitude)}$$

where Z = UTM Zone Number

Example UTM zone calculation

λ = 077° 08' 44.3456" W

Z = 17.14239

Z = 17 + 1

Z = 18

4-4. Geocentric Coordinate Systems

a. World Geodetic System of 1984 (WGS 84). The geocentric reference system used most often is the WGS 84. This is because WGS 84 is the reference ellipsoid for GPS. For most applications, the three-dimensional (3-D) geocentric coordinates of WGS 84 are converted into their equivalent two-dimensional (2-D) horizontal and vertical components and then used. For example, the geocentric coordinates are most often converted to 2-D NAD 83 SPCS horizontal coordinates and ellipsoid elevation values.

b. Ellipsoid coordinate system definition. Once the reference ellipsoid is defined, a 3-D (i.e., x, y, and z axis) Cartesian coordinate system also can be defined relative to the ellipsoid definition. Subsequent determination of a point relative to the reference ellipsoid and associated Cartesian coordinate system is made based on the difference in x, y, and z (i.e., $\triangle$x, $\triangle$y, and $\triangle$z) between the x, y, and z of the point and the references ellipsoid/Cartesian coordinate axes.

4-5. Geocentric Conversions

The primary use of geocentric coordinates is in conjunction with the use of GPS for the densification of military construction and civil works project control. Horizontal and vertical densification are handled as separate issues, therefore necessitating the conversion of the 3-D geocentric coordinates to 2-D horizontal components (i.e., x and y or latitude and longitude) and an elevation on a particular datum. Conversion of geocentric coordinates in this manner permits their use, for practical engineering purposes, directly on any local user datum. For example, GPS derived WGS 84 baselines in a network (i.e., geocentric coordinates) can be directly used on an NAD 27, NAD 83, or even a local project datum. Minor variation between these datums will be minimal when the GPS data used to formulate the baselines are used to adjust the fit between the local datum stations. Such assumptions are good for a localized project but may not be as valid when high-order NGRS network densification work is being performed.

a. Geodetic to Cartesian coordinate conversion. Given geodetic coordinates on NAD 83 (in ϕ, λ, H) or NAD 27, the geocentric Cartesian coordinates (X, Y, and Z) on the WGS 84, GRS 80, or Clarke 1866 ellipsoid can be converted directly by formulas as related in EM 1110-1-1003.

b. Cartesian to geodetic coordinate conversion. In the reverse case, given GRS 80 X, Y, Z coordinates, the conversion to NAD 83 geodetic coordinates (ϕ, λ, H) can be performed using a noniterative method (Soler and Hothem 1988) that is detailed in EM 1110-1-1003.

c. OCONUS Cartesian to geodetic coordinate conversion. Transformations between other OCONUS datums may be performed by changing the ellipsoidal parameters (i.e., *a*, *b*, and *f*) shown in EM 1110-1-1003 to the OCONUS datum's reference ellipsoid.

d. Example geographic to geocentric coordinate transformation. An example of a geographic to geocentric transformation can be found in EM 1110-1-1003.

e. Example geocentric to geographic transformation. An example of a geocentric to geographic transformation can be found in EM 1110-1-1003.

4-6. Horizontal Transformations NAD 27 to NAD 83

a. Conversion techniques. Any USACE survey control which was in the NGS database should have been converted to NAD 83 with corresponding results published by NGS. However, most USACE survey control, although referenced to NAD 27, was not in the NGS database and was not included in the NGS readjustment and redefinition of the national geodetic network. Therefore, the USACE will have to convert this control to NAD 83 by some mathematical adjustment or transformation process. All conversion methods are dependent on the existence of nearby survey controls that are, or can be, referenced to both the NGS NAD 83 database and the USACE survey control requiring conversion. These are referred to as common stations from which unknown or intermediate points are best fitted. The process is analogous to that of a simple traverse adjustment; e.g., if the fixed control points are changed, then a recomputation of the intermediate traverse stations causes a differential shift at each point proportionate to the overall change in the fixed coordinates. Although a variety of methods exist to perform these conversions, only the three that are considered applicable to USACE projects are discussed below.

(1) Survey or resurvey of projects using NAD 83 control. A new survey using NGS published NAD 83 control could be performed over an entire project. This could be either a newly authorized project or one undergoing major renovation or maintenance. A resurvey of an existing project must tie in all monumented points and may be performed using any conventional survey procedure. Although this is not a datum transformation technique, it accomplishes the same thing. A resurvey of an existing project would not normally be economically justified unless major renovation work is being performed or the existing NAD 27 control is of low density or accuracy. A resurvey of an existing project should not be performed solely to transform the project to NAD 83. One of the other methods described below should suffice. One disadvantage of this procedure is that other users of USACE survey data may also convert these data to NAD 83 using a mathematical transformation resulting in different coordinate values for the same survey points. If this method is employed, actions may have to be taken to notify other users of the USACE NAD 83 coordinates.

(2) Readjustment of original survey observations. If the original project control survey was connected to common NGS stations, the survey may be readjusted using NAD 83 coordinates instead of the NAD 27 coordinates originally used. This method involves locating the original field angular, distance, and azimuth observations, and completely readjusting the survey using a recognized adjustment method, such as a rigorous least-squares or other equally adequate semi-rigorous method (including traditional traverse adjustment methods). The original Corps survey scheme (e.g., traverse, triangulation, trilateration, etc.) must have been rigidly connected to the National Geodetic Control Network (i.e., published coordinates on both NAD 27 and NAD 83), which allows the readjustment process to be performed using only the published NAD 83 control as fixed. This procedure is extremely labor intensive and would be justified only on selected projects. Another disadvantage of this procedure is that other users of USACE survey data may also convert these data to NAD 83 using a mathematical transformation resulting in different coordinate values for the same survey points. It is, however, theoretically superior to the mathematical transformation methods described below. If this method is employed, actions may have to be taken to notify other users of the USACE NAD 83 coordinates.

(3) Mathematical transformations. Since neither of the above methods can be economically justified on many USACE projects, mathematical techniques for transforming project control data to NAD 83 have been developed. These methods yield results which are normally better than ± 1 foot of the actual values and errors are typically consistent within a project area. Therefore, they are considered approximate. However, this will probably meet most USACE needs. They should be used with caution when real property demarcation points are involved. When mathematical transformations are employed they should be adequately noted so that users will be aware of the conversion method and accuracy of the resulting coordinates.

(a) A 3-D coordinate transformation process from one geodetic reference system to another can generally be represented as a simple three-step process:

(1) Convert elliptical (geodetic or geographical) angular coordinate system to a 3-D rectangular (Cartesian) coordinate system.

(2) Translate and reorient the origin from the old ellipsoid/datum to the new ellipsoid/datum using known translation and orientation values.

(3) Convert the new Cartesian system to its geodetic ellipsoid coordinate reference system.

Performing the second step is not straightforward since the method of defining the datum origin and axis orientation parameters differs significantly between NAD 27 and NAD 83. Thus, a simple 3-D coordinate relationship does not exist between the two systems. As a result, local scale distortions exist between NAD 27 and NAD 83 and this distortion is known only at NGS published points common to both datums. Such NGS control which was originally published on NAD 27 and readjusted as part of the 1983 NAD redefinition are termed common points.

(b) A USACE survey control point will typically fall between these common points, and the distortion at this point due to the NAD 83 readjustment can only be estimated. Since coordinates are known for common points which were included in both the NAD 27 and NAD 83 adjustments, datum differences or shifts at these common points provide the necessary mathematical model to transform points which were not common to both datums. Such a coordinate transformation process is usually performed by a so-called rigorous mathematical minimization or regression technique. This is typically accomplished by some form of a least squares adjustment whereby the point to be transformed is best fitted between the surrounding common points. A rigorous transformation from NAD 27 to NAD 83 does account and compensate for not only the ellipsoidal orientation differences but also the local readjustment changes in the reference network. Any point which has been converted by such a transformation method should be considered as having approximate NAD 83 coordinates. This is because these coordinates were not computed using original survey observations.

b. Performing mathematical transformations. Numerous mathematical techniques have been developed to convert coordinates from NAD 27 to NAD 83. These include a variety of multiple parameter transformation equations and multiple regression transformation equations. Each technique has advantages and disadvantages in terms of accuracy, consistency, and complexity of the process.

(1) LEFTI. One of these was the computer program LEFTI, developed by NGS and distributed within USACE by the U.S. Army Topographic Engineering Center (TEC) in 1987 within the computer program DATATRAN. The results from LEFTI are dependent upon the selection of a set of points which are common to both NAD 27 and NAD 83. Selecting different common points yields different NAD 83 coordinates of the other points being converted. As various conversions are performed with LEFTI, different coordinates will be computed for the same point. Although these differences are small, they create inconsistency in survey data which could lead to intolerable situations for many applications.

(2) NADCON/CORPSCON. To eliminate these inconsistencies, NGS developed the transformation program NADCON which does not require selection of common points and yields consistent results. This technique is based on a bi-harmonic equation classically used to model plate deflections. NADCON has been reconfigured by TEC into a more comprehensive program called CORPSCON, which also converts between SPCS 27, SPCS 83, UTM 27, and UTM 83; thus eliminating several steps in the total process. CORPSCON will be the standard conversion method for USACE, replacing the previously used LEFTI. Technical documentation and operating instructions for CORPSCON are contained in Appendix D.

c. Distortion of dimensions. In addition to changing all coordinates on a project, the conversion process could slightly modify project dimensions. Localized coordinate changes created by the 1983 readjustment could result in very slight distortions of project dimensions and alignment data. This is due to the varying amount of datum shift from point to point. This variation could be significant over small distances typically encountered on USACE projects. For this reason, care must be taken in the use of mathematical transformations to convert existing control to NAD 83, whether it be fixed monumented points or project alignment data, e.g., PIs, PTs, and all other similar computed or nonoccupied points. As stated earlier, this variable shift is due to both the readjustment and redefinition of the NAD 83 reference datum. In addition, the various SPCS 83 grid references were redefined with origins deliberately changed by large amounts to avoid confusion with the older SPCS 27. When transformed NAD 83 geographic coordinates are subsequently converted to redefined SPCS 83 grid coordinates, the total magnitude of the coordinate shift between SPCS 27 and SPCS 83 contains both the geographic datum shift (due to redefinition and readjustment) and the local grid redefinition shift.

(1) Shift gradients. Since the overall datum shift varies from point to point throughout North America, the amount of datum shift across a local project is not constant. The variation can be as much as 0.1 foot per mile. Some typical coordinate shift variations that can be expected over a 10,000-foot section of a project are shown below:

Project Area	SPCS Reference	per 10,000 feet
Baltimore, MD	1900	0.16 ft
Los Angeles, CA	0405	0.15 ft
Mississippi Gulf Coast	2301	0.08 ft
Mississippi River (IL)	1202	0.12 ft
New Orleans, LA	1702	0.22 ft
Norfolk, VA	4502	0.08 ft
San Francisco, CA	0402	0.12 ft
Savannah, GA	1001	0.12 ft
Seattle, WA	4601	0.10 ft

(2) Example. Such local scale changes will cause project alignment data to distort by unequal amounts. Thus, a 10,000.00-foot tangent on 1927 NAD project coordinates could end up as 9,999.91 feet after a mathematical transformation of the PI stations to NAD 83 coordinates. Although such variances may not be physically significant from a construction standpoint, the potential for such numerical anomalies must be recognized. Therefore, in addition to significantly changing all absolute coordinates on a project, the datum transformation process will slightly modify the project's design dimensions and/or construction orientation and scale. On a navigation project, for example, an 800.00-foot-wide channel could

vary from 799.98 to 800.04 feet along its reach. The grid azimuth between the PIs will also change. If the local SPCS 83 grid was modified, then even larger dimension changes can result.

(3) Corrections. Correcting this may require recomputation of coordinates after conversion to ensure that original project dimensions and alignment data remain intact. This is particularly important for property and boundary surveys. Alternatively, a fixed shift may be applied to all data points over a limited area. Determining the maximum area over which such a fixed shift can be applied is important.

(4) Computing fixed conversion factors. CORPSCON may be employed to compute a conversion factor to within ±1 foot. This conversion could be held fixed and applied to all coordinates over a given sector of a project or an entire project. Typically, this fixed conversion would be computed at the center of a sheet or at the center of a project and the conversions in X and Y from NAD 27 to NAD 83 and from SPCS 27 to SPCS 83 indicated by notes on the sheets or data sets. Since the conversion is not constant over a given area, the fixed conversion amounts must be explained in a note. The magnitude of the conversion factor gradient across a sheet is a function of location and the drawing scale. Whether the magnitude of the gradient is significant depends on the nature of the project. A 0.5-foot variation on an off-shore navigation project may be acceptable for converting depth-sounding locations, whereas a 0.1-foot gradient may be intolerable for construction layout on an installation. In any event, the magnitude of this gradient should be computed by CORPSCON at each end (or corners) of a sheet or project. If the conversion factor variation exceeds the allowable tolerances, then a fixed conversion factor should not be used. Examples of two fixed conversion factors follow:

(a) Let's assume we have a 1" = 40' scale site plan map on existing SPCS 27 (VA South Zone 4502). Using CORPSCON, convert existing SPCS 27 coordinates at the sheet center and corners to SPCS 83 (U.S. Survey Foot), and compare SPCS 83 and 27 differences.

	SPCS 83	SPCS 27	SPCS 83 - SPCS 27
Center	N 3,527,095.554	Y 246,200.000	dY = 3,280,895.554
of sheet	E 11,921,022.711	X 2,438,025.000	dX = 9,482,997.711
NW	N 3,527,595.553	Y 246,700.000	dY = 3,280,895.553
corner	E 11,920,522.693	X 2,437,525.000	dX = 9,482,997.693
NE	N 3,527,595.556	Y 246,700.000	dY = 3,280,895.556
corner	E 11,921,522.691	X 2,438,525.000	dX = 9,482,997.691
SE	N 3,526,595.535	Y 245,700.000	dY = 3,280,895.535
corner	E 11,921,522.702	X 2,438,525.000	dX = 9,482,997.702
SW	N 3,526,595.535	Y 245,700.000	dY = 3,280,895.535
corner	E 11,920,522.704	X 2,437,525.000	dX = 9,482,997.704

Since coordinate differences do not exceed 0.03 foot in either the X or Y direction, the computed SPCS 83-27 coordinate differences at the center of the sheet may be used as a fixed conversion factor to be applied to all existing SPCS 27 coordinates on this drawing.

(b) Assuming a 1" = 1,000' base map is prepared of the same general area, a standard drawing will cover some 30,000 feet in an east-west direction. Computing SPCS 83-27 differences along this alignment yields the following:

	SPCS 83	SPCS 27	SPCS 83 - SPCS 27
West	N 3,527,095.554	Y 246,200.000	dY = 3,280,895.554
end	E 11,921,022.711	X 2,438,025.000	dX = 9,482,997.711
East	N 3,527,095.364	Y 246,200.000	dY = 3,280,895.364
end	E 11,951,022.104	X 2,468,025.000	dX = 9,482,997.104

The conversion factor gradient across this sheet is about 0.2 foot in Y and 0.6 foot in X. Such small changes are not significant at the plot scale of 1" = 1,000'; however, for referencing basic design or construction control (either monumented points or within an Intergraph Design File), applying a fixed shift across an area of this size is not recommended -- individual points should be transformed separately. If this 30,000-foot distance were a navigation project, then a fixed conversion factor computed at the center of the sheet would suffice for all bathymetric features.

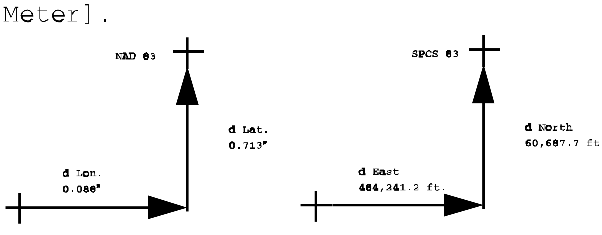
(5) Redefined SPCS 83 grid parameters. In developing the SPCS 83 grid system, some states made significant modifications relative to their previous SPCS 27 grid. As a result, a fixed conversion factor cannot be performed by simply translating the **X** and **Y** differences between SPCS 27 and SPCS 83 -- a grid rotational component may also be present if the central meridian was changed. Variation in the *CORPSCON* computed convergence for both SPCS 27 and 83 is an indication of this. For those states/zones which have been modified, and excessive shift differences occur over small distances, then a fixed conversion factor for even a small area or sheet is not appropriate. A geographic datum shift (in seconds of arc) may still be used in these cases provided the variation over the project/sheet is not significant.

d. Project continuity. Caution should be exercised when converting portions of projects or military installations or projects that are adjacent to other projects that may not be converted. If the same monumented control points are used for several projects or parts of the same project, different datums for the two projects or parts thereof could lead to surveying and mapping errors, misalignment at the junctions, and layout problems during construction.

e. Metric units. SPCS 83 is defined in metric units. Therefore, coordinate conversions between geographic positions (latitude and longitude) and SPCS 83 use metric units. However, since most USACE work will continue in feet, conversion to feet will be required. The FGCC recommended standard for Federal surveying applications in the United States is still the U.S. Survey Foot (1200/3937 Meters). USACE has traditionally used the U.S. Survey Foot and will continue to do so in most instances. However, since some states have adopted the International Foot (30.48/100 meters) and others may choose to use metric units, property and boundary surveys in these states may require this conversion factor, or use of metrics, to be legally correct.

f. Datum notes. During the period of conversion to NAD 83 there will be two datums in use. Therefore, a note must be added to all maps, engineering site drawings, and documents containing coordinate information that indicates which datum is being used. If the control has been converted from NAD 27 to NAD 83, identify the conversion method used. A datum note should be repeated on separate sheets as opposed to a general note on the index or cover sheet. The note will be as follows:

```
The coordinates shown are referenced
to NAD [27/83] and are in feet based
on the SPCS [27/83] [state, zone].
Differences between NAD 27 and NAD
83 and between SPCS 27 and SPCS 83
at the center of the [sheet/data
set] are shown on the diagram below.
Datum conversion was performed using
the computer program "CORPSCON."
Metric conversions were based on [US
Survey Foot = 1200/3937 Meter] or
[International Foot = 30.48/100
Meter].
```



Note: Do not reference SPCS 27-83 differences if the SPCS 83 grid was significantly redefined. In such cases, omit the SPCS 27-83 differences and show only the NAD 27-83 differences in arc seconds of latitude and longitude.

g. Dual grids ticks and coordinate systems. Depicting both NAD 27 and NAD 83 grid ticks and coordinate systems on maps and drawings should be avoided where possible. This is often confusing and can increase the chance for errors during design and construction. However, where use of dual grid ticks and coordinate systems is unavoidable, only secondary grid ticks in the margins will be permitted.

h. Global Positioning System (GPS). GPS surveying techniques and computations are based on WGS 84, which are consistent with NAD 83. Differential (static) GPS surveying techniques are accurate for high order control over very large distances.

(1) Problem. If GPS is used to set new control points referenced to higher order control many miles from the project, inconsistent data may result at the project site. If the new control is near older control points which have been converted to NAD 83, two slightly different networks can result, even though both have NAD 83 coordinates. This slight difference creates problems in adjusting the data and using the two networks as one.

(2) Solution. In order to avoid this situation, locate the GPS base stations on the control in the project area, i.e., don't transfer it in from outside the area. Use the CORPSCON program to convert the old control from NAD 27 to NAD 83 and use these NAD 83 values to initiate the GPS survey. This allows GPS to produce coordinates that are both referenced to NAD 83 and consistent with the old control.

i. Local project datums. Local project datums which are not referenced to NAD 27 cannot be mathematically converted to NAD 83. Field surveys connecting them to other stations that are referenced to NAD 83 are required. GPS is the most economical and accurate technique to accomplish this connection.

4-7. Horizontal Transition Plan

a. General. Not all maps, engineering site drawings, documents, and associated products containing coordinate information will require conversion to NAD 83. To ensure that an orderly and timely transition to NAD 83 is achieved for the appropriate products, the following general guidelines should be followed:

(1) All initial surveys should be referenced to NAD 83.

(2) Active projects where maps, site drawings, or coordinate information are provided to non-USACE users (e.g., NOAA, USCG, FEMA, and others in the public and private sector) should be converted to NAD 83 the next time the project is surveyed or maps or site drawings are updated for other reasons.

(3) For in-active projects or active projects where maps, site drawings, or coordinate information are not normally provided to non-USACE users, conversion to NAD 83 is optional.

(4) Whenever maps, site drawings, or coordinate information (regardless of the above type) are provided to non-USACE users, it should contain a datum note.

b. Levels of effort. For maps and site drawings the conversion process entails one of three levels of effort: conversion of coordinates of all mapped details to NAD 83 and redrawing the map, replace the existing map grid with a NAD 83 grid, and simply adding a datum note. For surveyed points, control stations, alignment, and other coordinated information, conversion must be through either a mathematical transformation or readjustment of survey observations.

c. Detailed instructions.

(1) Initial surveys on civil works projects. The project control should be established on NAD 83 relative to the NGRS using conventional or GPS surveying procedures. The local SPCS 83 grid should be used on all maps and site drawings. All planning and design activities should then be based on the SPCS 83 grid. This includes supplemental site plan mapping, core borings, project design and alignment, construction layout and payment surveys, and applicable boundary or property surveys. All maps and site drawings shall contain datum notes. If the local sponsor requires the use of NAD 27 for continuity with other projects which they have not yet converted to NAD 83, conversion to NAD 27 could be performed using the *CORPSCON* transformation techniques described in Appendix D.

(2) Active civil works O&M projects undergoing maintenance or repair. These projects should be converted to NAD 83 during the next maintenance or repair cycle in the same manner as for newly initiated civil works projects. However, if resources are not available for this level of effort, either redraw the grids or add the necessary datum notes. Plans should be made for the full conversion during a later maintenance or repair cycle when resources can be made available.

(3) Military construction and master planning projects. All MCA, OMA, AFH, OMAR, MCAF, OMAF, and master planning projects will remain on NAD 27 or the current local datum until a thoroughly coordinated effort can be arranged with the MACOM DCSENGR and installation DEH or AFRCE and BCE. An entire installation's control network should be transformed simultaneously to avoid different datums on the same installation. The respective MACOMS are responsible for this decision. However, military operations may require NAD 83, including SPCS 83 or UTM metric grid systems. If so, these shall be performed separate from facility engineering support. A dual grid system may be required for such operational applications when there is overlap with normal facilities engineering functions. Coordinate transformations throughout an installation can be computed using the procedure described in Appendix D. Care must be taken when using transformations from NAD 27 with new control set using GPS methods from points remote from the installation. Installation boundary surveys should adhere to those outlined under real estate surveys listed below.

(4) Real estate.

(a) Surveys, maps, and plats prepared in support of civil works and military real estate activities should conform as much as possible to state requirements. Since most states have adopted NAD 83, most new boundary and property surveys should be based on NAD 83. The local authorities should be contacted before conducting boundary and property surveys to ascertain their policies.

(b) It should be noted that several states have adopted the International Foot for their standard conversion from meters to feet. However, recent action by the FGCS to affirm continued use of the U.S. Survey Foot for Federal surveying activities could lead to a reversal of some state policies.

(c) In order to avoid dual coordinates on USACE survey control points which have multiple uses, all control should be based on the U.S. Survey Foot, including control for boundary and property surveys. In states where the International Foot is the only accepted standard for boundary and property surveys, conversion of these points to NAD 83 should be based on the International Foot, while the control remains based on the U.S. Survey Foot.

(5) Regulatory functions. Surveys, maps, and site drawings prepared in support of regulatory functions should begin to be referenced to NAD 83 unless there is some compelling reason to remain on NAD 27 or locally used datum. Conversion of existing surveys, maps, and

drawings to NAD 83 is not necessary. Existing surveys, maps, and drawings need only have the datum note added before distribution to non-USACE users. The requirements of local, state, and other Federal permitting agencies should be ascertained before site-specific conversions are undertaken. If states require conversions based on the International Foot, the same procedures as described above for real estate surveys should be followed.

(6) Other existing projects. Other existing projects, e.g., structural deformation, beach nourishment, submerged offshore disposal areas, historical preservation projects, etc., need not be converted to NAD 83. However, existing surveys, maps, and drawings should have the datum note added before distribution to non-USACE users.

(7) Work for others. Existing projects for other agencies will remain on NAD 27 or the current local datum until a thoroughly coordinated effort can be arranged with the sponsoring agency. The decision to convert rests with the sponsoring agency. However, existing surveys, maps, and drawings should have the datum note added before distribution to non-USACE users. If sponsoring agencies do not indicate a preference for new projects, NAD 83 should be used. The same procedures as described above for initial surveys on civil works projects should be followed (see paragraph *c*(1)).

4-8. Vertical Datums

A vertical datum is the surface to which elevations or depths are referred to or referenced. There are many vertical datums in the CONUS. The surveyor should be aware of the vertical control datum being used and its practicability to meet project requirements. The following paragraphs will detail further some of the vertical control systems and associated datums used in the CONUS.

a. NGVD 29. NGVD 29 was established by the United States Coast and Geodetic Survey's (USC&GS's) 1929 General Adjustment. NGVD 29 was established by constraining the combined U.S. and Canadian first order leveling nets to conform to MSL as determined at 26 long-term tidal gage stations that were spaced along the east and west coasts of North America and along the Gulf of Mexico, with 21 stations in the U.S. and 5 stations in Canada. The NGVD 29 was originally named the Mean Sea Level Datum of 1929. It was known at the time that because of the variation of ocean currents, prevailing winds, barometric pressures, and other physical causes, the MSL determinations at the tide gages would not define a single equipotential surface. The name of the datum was changed from the Mean Sea Level Datum to the NGVD 29 in 1973 to eliminate the reference to sea level in the title. This was a change in name only; the definition of the datum established in 1929 was not changed. Since NGVD 29 was established, it has become obvious that the geoid based upon local mean tidal observations would change with each measurement cycle. Estimating the geoid based upon the constantly changing tides does not provide the most stable estimate of the shape of the geoid, or the basic shape of the earth.

b. NAVD 88. NGVD 29 has been replaced by NAVD 88, an international datum adopted for use in Canada, the United States, and Mexico. NAVD 88 was established to resolve problems and discrepancies in NGVD 29. The datum for NAVD 88 is based upon the mass or density of the earth instead of the varying heights of the seas. Measurements in the acceleration of gravity are made at observation points in the network, and only one datum point, at Pointe-au-Pere/Rimouski, Québec, Canada, is used. The vertical reference surface is therefore defined by the surface on which the gravity values are equal to the control point value. The result of this adjustment is newly published NAVD 88 elevation values for benchmarks (BMs) in the NGS inventory. Most Third-Order BMs, including those of other Federal, state, and local government agencies, were not included in the NAVD 88 adjustment. The FGCS of the Federal Geographic Data Committee (FGDC) has affirmed that NAVD 88 shall be the official vertical reference datum for the U.S. The FGDC has prescribed that all surveying and mapping activities performed or financed by the Federal Government make every effort to begin an orderly transition to NAVD 88, where practicable and feasible.

c. MSL datums. Some vertical datums are referenced to MSL. Such datums typically are maintained locally or within a specific project area. The theoretical bases for these datums are just as their title indicates: the mean sea level. Local MSL is a vertical datum of reference that is based upon the observations from one or more tidal gaging stations. NGVD 29 was based upon the assumption that local MSL at those 21 tidal stations in the U.S. and 5 tidal stations in Canada equaled 0.0000 foot on NGVD 29. The value of MSL as measured over the Metonic cycle of 19 years shows that this assumption is not entirely valid and that MSL varies from station to station.

d. Tidal datums. Some vertical datums are referenced to tidal waters or lake levels. An example of a lake-level-based tidal datum used as a vertical datum is the International Great Lakes Datum of 1955 (IGLD 55), maintained and used for vertical control in the Great Lakes region of the CONUS. Just like the NGVD, these datums undergo periodic adjustment. For example, the IGLD 55 was adjusted in 1985 to produce IGLD 85. Unlike the prior datum (IGLD 55), IGLD 85 has been directly referenced to NAVD 88 and originates at the same point as NAVD 88. Tidal datums typically are defined by the phase of the tide and usually are described as MHW, MLW, and MLLW with each description having a particular relevance to the datum definition. For further information on these and other tidal-datum-related terms, the reader is advised to refer to any of the applicable texts in Appendix A.

e. Other vertical datums. Other areas of the CONUS and OCONUS may maintain and employ other vertical datums. For instance, there are a variety of vertical datums maintained in Alaska, Puerto Rico, Hawaii, the Virgin Islands, Guam, and other islands and project areas. Specifications and other information for these particular vertical datums can be obtained from the particular FOA responsible for survey-related activities for these areas or the National Ocean Service (i.e., NOS).

4-9. Distinction Between Orthometric and Dynamic Heights

a. There are several different reference elevation systems used by the surveying and mapping community. Two of these height systems are relevant to IGLD 85: orthometric heights and dynamic heights. Geopotential numbers relate these two systems to each other. The geopotential number (C) of a BM is the difference in potential measured from the reference geopotential surface to the equipotential surface passing through the survey mark. In other words, it is the amount of work required to raise a unit mass of 1 kg against gravity through the orthometric height to the mark. Geopotential differences are differences in potential which indicate hydraulic head. The orthometric height of a mark is the distance from the reference surface to the mark, measured along the line perpendicular to every equipotential surface in between. A series of equipotential surfaces can be used to represent the gravity field. One of these surfaces is specified as the reference system from which orthometric heights are measured. These surfaces defined by the gravity field are not parallel surfaces because of the rotation of the earth and gravity anomalies in the gravity field. Two points, therefore, could have the same potential but may have two different orthometric heights. The value of orthometric height at a point depends on all the equipotential surfaces beneath that point.

b. The orthometric height (H) and the geopotential number (C) are related through the following equation:

$$C = G * H$$

where G is the gravity value estimated for a particular system. Height systems are called different names depending on the gravity value (G) selected. When G is computed using the Helmert height reduction formula that is used for NAVD 88, the heights are called Helmert Orthometric Heights. When G is computed using the International Formula for Normal Gravity, the heights are called Normal Orthometric Heights. When G is equal to normal gravity at 45^{o} latitude, the heights are called Normal Dynamic Heights. It should be noted that dynamic heights are just geopotential numbers scaled by a constant, using normal gravity at 45^{o} latitude equal to 980.6199 gal. Therefore, dynamic heights are also an estimate of hydraulic head. In other words, two points that have the same geopotential number will have the same dynamic height.

c. IGLD 55 is a normal dynamic height system which used a computed value of gravity based on the International Formula for Normal Gravity. Today, there is sufficient observed gravity data available to estimate “true” geopotential differences instead of “normal” geopotential differences. The “true” geopotential differences, which were used in developing IGLD 85 and NAVD 88, will more accurately estimate hydraulic head.

4-10. Vertical Transformations NGVD 29 to NAVD 88

For map and site plan drawings, including digital variations thereof, the conversion process entails one of two levels of effort: (1) conversion of all elevations to NAVD 88 and redrawing the map, or (2) simply adding a datum note based on an approximate VERTCON conversion.

a. VERTCON is a program developed by the NGS that converts elevation data from NGVD 29 to NAVD 88. VERTCON uses a simplified transformation of BM heights using a modeled shift for a given area and is, in general, only sufficiently accurate to meet small-scale mapping requirements. VERTCON should not be used for converting BM elevations used for site plan design or construction applications. Users can simply input the latitude and longitude for a point and the vertical datum shift between NGVD 29 and NAVD 88 is output. VERTCON returns the orthometric height difference between

NAVD 88 and NGVD 29 at the geodetic position specified by the user. The root-mean-square (RMS) error of the actual NGVD 29/NAVD 88 height differences versus the computed height differences from the model for the data points used to create the model is ±1 cm; the estimated maximum error is ±2.5 cm. Depending on network design and terrain relief, larger differences (e.g., 5 to 50 cm) may occur the further a benchmark is located from the control points used to establish the model coefficients. For this reason, VERTCON should only be used for approximate conversions where these potential errors are not critical.

b. Whenever maps, site drawings, or spatial elevation data are provided to non-USACE users, they should contain a datum note that provides, at minimum, the following information:

> The elevations shown are referenced to the [NGVD 29] [NAVD 88] and are in [feet] [meters]. Differences between NGVD 29 and NAVD 88 at the center of the project sheet/data set are shown on the diagram below. Datum conversion was performed using the [program VERTCON] [direct leveling connections with published NGS benchmarks] [other]. Metric conversions are based on [U.S. Survey Foot = 1200/3937 meters] [International Survey Foot = 30.48/100 meters].

c. There are several compelling reasons that make it advantageous for USACE commands to convert to NAVD 88. These include:

- Differential leveling surveys between benchmarks will often close better.
- NAVD 88 will provide a better reference to estimate GPS-derived orthometric heights.
- IGLD 85 will provide a better reference to estimate heights of water-level surfaces on the Great Lakes.
- Data and adjusted height values will be readily available and accessible in convenient form from NGS's Integrated Data Base.
- Federal surveying and mapping agencies will stop publishing on NGVD 29 and IGLD 55, and will publish only on NAVD 88 and IGLD 85.
- Surveys performed for the Federal government will require use of NAVD 88.
- NAVD 88 is recommended by ACSM and FGCS.

4-11. Vertical Transition Plan

A change in vertical datum will affect most USACE engineering, construction, planning, and surveying activities. The cost of conversion could be substantial at the onset. There is a potential for errors in conversions inadvertently occurring. The effects of the vertical datum change can be minimized if the change is gradually applied over time; being applied to future projects and efforts, rather than concentrated on changing already published products. In order to ensure an orderly and timely transition to NAVD 88 is achieved for the appropriate products, the following general guidelines in this section should be followed.

a. Conversion criteria. Maps, engineering site drawings, documents, and associated spatial data products containing elevation data may require conversion to NAVD 88. Specific requirements for conversion will, in large part, be based on local usage -- that of the local sponsor, installation, etc. Where applicable and appropriate, this conversion should be recommended to local interests.

b. Newly authorized construction projects. Generally, initial surveys of newly authorized projects should be referenced to NAVD 88. In addition to design/construction, this would include wide-area master plan mapping work. The project control should be referenced to NAVD 88 using conventional or GPS surveying techniques. All planning and design activities should be based upon NAVD 88. All maps and site drawings shall contain datum notes as described below. If the sponsor/installation requires the use of NGVD 29 or some other local vertical reference datum for continuity, the relationship between NGVD 29 and NAVD 88 shall be clearly noted on all maps, engineering site drawings, documents, and associated products.

c. Active projects. On active projects where maps, site drawings, or elevation data are provided to non-USACE users, the conversion to NAVD 88 should be performed. This conversion to NAVD 88 may be performed the next time the project is surveyed or when the maps/site drawings are updated for other reasons. Civil works projects may be converted to NAVD 88 during the next maintenance or repair cycle in the same manner as that for newly initiated civil works projects. However, if resources are not available for this level of effort, either redraw the maps or drawings and add the necessary datum note. Plans should be made for the full conversion during a later maintenance or repair cycle when resources can be made available. MCA, OMA, AFH, OMAR, MCAF, OMAF and master planning projects should remain on NGVD 29 or the local vertical datum until a thoroughly coordinated effort can be arranged with the MACOM DCSENGR and installation DEH, or AFRCE and BCE. An entire installation's control network should be transformed simultaneously to avoid different datums on the same installation. MACOMs should be encouraged to convert to NAVD 88. However, the respective MACOMs are responsible for this decision.

d. Inactive projects. For inactive projects or active projects where maps, site drawings, or elevation data are not normally provided to non-USACE users, conversion to NAVD 88 is optional.

e. Work for others. Projects for other agencies will remain on NGVD 29 or the current local vertical datum until a thoroughly coordinated effort can be arranged with the sponsoring agency. Other agencies should be encouraged to convert their projects to NAVD 88, although the decision to convert rests with the sponsoring agency. However, surveys, maps, and drawings should have a datum note added before distribution to non-USACE users. If sponsoring agencies do not indicate a preference for new projects, NAVD 88 should be used.

f. Miscellaneous projects. Other projects referenced to strictly local datum, such as structural deformation, beach nourishment, submerged offshore disposal areas, historical preservation projects, etc., need not necessarily be converted to NAVD 88. However, it is recommended that surveys, maps, and drawings have a clear datum reference note added before distribution to non-USACE users.

g. Real estate implications.

(1) Surveys, maps, and plats prepared in support of civil works and military real estate activities should conform as much as possible to state requirements. Many states are expected to adopt NAVD 88 (by statute) as an official vertical reference datum. This likewise will entail a transition to NAVD 88 in those states. State and local authorities should therefore be contacted to ascertain their current policies.

(2) Note that several states have adopted the International Foot for their standard conversion from meters to feet. However, recent action by the FGCS to affirm continued use of the U.S. Survey Foot for Federal surveying activities could lead to a reversal of some state policies.

(3) In order to avoid dual elevations on USACE survey control points which have multiple uses, it is recommended that published elevations be based on the U.S. Survey Foot. In states where the International Foot is the only accepted standard for boundary and property surveys, conversion of these elevations to NAVD 88 should be based on the International Foot while the control remains based on the U.S. Survey Foot.

Chapter 5
Horizontal Control Survey Techniques

5-1. General

This chapter outlines procedures to follow for horizontal control surveys. The material presented is not meant to be rigid methodology that cannot be tailored for a particular application. The mandatory classifications and standards portions must be followed, while the recommendations should be followed whenever possible. Refer to EM 1110-1-1003 for more information on GPS survey techniques, as well as any of the applicable references listed in Appendix A for further guidance on information not covered in this chapter.

NOTE: The accuracy of control surveying measurements should be consistent with the purpose of the survey. It is important to remember that the best survey is the one that provides the data at the required accuracy levels without wasting manpower, time, and money.

5-2. Horizontal Control Surveys

Horizontal control surveys are done to establish primary or secondary horizontal control points. The procedures to establish the control are traverse, triangulation, and/or trilateration. The preferred method for establishing horizontal control is to use a traverse and GPS survey techniques. When conventional horizontal control survey techniques are used, the preferred instrument is a total station; and if such equipment is not available, a theodolite and electronic distance measuring equipment (EDM) are often used. Always keep the project requirements paramount (e.g., accuracy, time, money, manpower, etc.) when determining what survey procedures to follow and the equipment to be used.

5-3. Primary Horizontal Control

Primary horizontal control is established at accuracies where practicable to meet the requirements of the application, typically at Third-Order Class I or better. Primary 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.

a. The minimum instrument requirements for the establishment of primary control shall be: a repeating theodolite with an optical micrometer with a least count resolution of six seconds (i.e., 6") or better; or a directional theodolite with an optical micrometer with a least count resolution of one second (i.e., 1") and an EDM capable of a resolution of 1:10,000; or a total station or other instrument having capabilities comparable to or better than any of the instruments just detailed.

b. Primary horizontal control points not permanently monumented in accordance with criteria and guidance established in EM 1110-1-1002 should meet the following minimum standards:

(1) Primary horizontal control points shall be marked with semipermanent-type markers (e.g., rebar, railroad spikes, large spikes).

(2) Primary horizontal control points shall be placed either flush with the existing ground level or buried a minimum of one tenth of a foot below the surface.

(3) Each primary control point shall be referenced by a minimum of two points to aid in future recovery of that point. For this reference, well defined natural or man-made objects may be used. The reference point(s) can be either set or existing and should be within 100 feet of the control point.

(4) A sketch shall be placed in a standard field book. The sketch at a minimum will show the relative location of each control point to the reference points and major physical features within 100 feet of the point.

(5) If concrete monuments are required, they will be established prior to the accomplishment of any horizontal survey work. These monuments will be established in accordance with EM 1110-1-1002.

c. Primary horizontal control points shall be occupied by an electronic total station, a theodolite and EDM, or comparable equipment. Establishing primary control points by one angle and one distance will not be permitted.

d. Distance measurements for primary horizontal control points shall be accomplished with a total station, an EDM, or other comparable equipment capable of obtaining an accuracy of 1:10,000.

e. When an EDM is used, a minimum of two readings shall be taken at each setup and recorded in a standard field book. The leveled height of the instrument and the height of the reflector shall be measured carefully to within 0.02 foot and recorded in the field book. Each slope distance shall be reduced to a horizontal distance using either reciprocal vertical angle observations or from the elevation of each point obtained using differential leveling.

f. All total stations, EDMs, and prisms used for primary control work shall be serviced regularly and checked frequently over lines of known length. Calibration should be done at least annually.

g. If a repeating theodolite (e.g., 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 shall then be turned to the right, and the first angle recorded in a field book. The angle shall 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 (i.e., 5") from the result of the last angle divided by four, the process shall 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 shall be closed by repeating this process for all of the sights to be observed from that location. The foresight for the last observation shall 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 (i.e., 10") from 360 degrees, the angles shall be turned again until the summation is within this tolerance.

h. If a directional theodolite (e.g., Wild T2, Wild T3) is used for the horizontal angles, the instrument shall be pointed at the backsight station with the telescope in a direct reading position and the horizontal vernier set to within ten seconds (i.e., 10") of zero degrees. The vernier shall be brought into coincidence and the angle read and recorded in the field book. The angles shall 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 shall include all sights to be observed from that station. The telescope shall then be inverted and the process repeated in reverse order, except the vernier is not to be reset, but will be read where it was originally set. The actual angles between stations may then be computed by differencing the direct and reverse readings. This process shall be repeated three times for a total of three data set collections.

i. Regardless of the theodolite used, at least once a year and whenever the difference between direct and reverse reading of any theodolite deviates more than 30 seconds from 180 degrees, the instrument should be adjusted for collimation error. Readjustment of the cross hairs and the level bubble should be done whenever their misadjustments affect the instrument reading by more than the least count of the vernier of the theodolite.

j. If a total station is used for the horizontal angles, the same procedure shall be followed as detailed in the preceding paragraphs *g* and *h*, depending on the type of total station used. When using high precision total stations, only half as many readings are generally required (two data set collections).

k. To reduce slope distances to horizontal, a vertical angle observation must be obtained from each end of each line being measured. The vertical angles shall 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 levels, this vertical angle requirement is not necessary.

l. All targets established for backsights and foresights shall be set directly over the point to be measured to. 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 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) will not be used to set primary control targets.

5-4. Secondary Horizontal Control

Secondary horizontal control is established at accuracies where practicable to meet the requirements of the application, generally at Third Order Class II or lower. Secondary horizontal control typically is established to determine the location of structure sections, cross sections, or topographic surface, or to pre-mark requirements for small- to medium-scale photogrammetric mapping.

a. Secondary horizontal control requirements are identical to those 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.

(2) Occupation. Secondary horizontal control points can be established by one angle and one distance.

(3) When a total station or EDM is used, a minimum of two readings shall be taken at each setup and recorded in a standard field book.

(4) If a repeating theodolite is used for the horizontal angles, the angle measurement shall be repeated a minimum of two times by alternating the telescope and pointing in the direct and inverted positions.

(5) If a directional theodolite is used for the horizontal angles, the process (described for primary control) shall be repeated two times for a total of two data set collections.

5-5. Traverse

A traverse is defined as the measurement of the lengths and directions of a series of straight lines connecting a series of points on the earth. The points connected by the lines of traverse are known as traverse stations. The measurements of the lengths and directions are used to compute the relative horizontal positions of these stations. In the past, the traverse distances were determined by indirect measurement or taping and these traverses were used primarily as supplementary control to area triangulation networks. However, with the advent of total stations and EDM's, the traverse is now frequently used for basic area control surveys. In addition to the observation of horizontal directions between traverse stations, the elevation of the stations must be determined by either direct or trigonometric leveling. Astronomic observations must be made along a traverse at prescribed intervals to control the azimuth of the traverse and sometimes to determine the deviation of the vertical (plumb line). The interval and type of astronomic observation will depend upon the order of accuracy required and the traverse methods used.

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

(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 in the field measurements. There are two types of closed traverses.

(a) Loop traverse. A loop traverse starts on a station of known position and terminates on the same station. A loop traverse will detect blunders and accidental errors, but will not disclose systematic errors or inaccuracies in the starting information.

(b) Connecting traverse. A connecting traverse starts on a station of known position and terminates on a different station of known position. When using this type of traverse the systematic errors and position inaccuracies can be detected and eliminated along with blunders and accidental errors. This type of traverse is always preferred over a loop traverse when a choice exists.

(2) Open traverse. The open traverse starts on a station of known position and terminates on an unknown position. In this type of 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. If conditions require it, a picture point may be located by a side shot (open traverse) if project specifications permit.

b. Guidelines. The following minimum guidelines should be followed when performing traverse procedures:

(1) Origin. All traverses will originate from and tie into an existing control line of equal or higher accuracy. No traverse will be stubbed off or dead-ended (e.g. radial or spur spots) except by specific instructions from the appropriate authority.

(a) Astronomic observation. Under some conditions it may not be possible to start or terminate on stations of known position and/or azimuth. In these cases, an astronomic observation for position and/or azimuth must be accomplished. For Third Order, only an astronomic azimuth is required at intervals along the traverse and at abrupt changes in direction of the traverse. Placement of these astronomic stations is governed by the order of accuracy required.

(b) Route and/or location. Generally, where to start a traverse and tie it in are predetermined. Where practicable, always show additional ties to other lines if the traverse course being followed approaches other lines or triangulation points. The advantage of additional ties is the possibility of isolating an error even if it is not the intention to adjust the traverse into the extra ties. The specific route of a new traverse shall be selected with care, keeping in mind its primary purpose and the possibility of its future use. Angle points should be set in protected locations if possible. Examples of protected locations include at fence lines, under communication or power lines near poles, or any permanent concrete structure. It may be necessary to set critical points underground. If so, reference the point relative to permanent features by a sketch, as buried points are often difficult to recover at future dates.

(2) Accuracy. Traverses are performed under four general orders of accuracy: First, Second, Third, or Construction Layout/Fourth Order. First-Order traverse work requires a high degree of accuracy, instruments, and methods of precision, and therefore typically is not done in the USACE. The order of accuracy for any traverse is determined by the equipment and methods used in the traverse measurements, by the accuracy attained, and by the accuracy of the starting and terminating stations of the traverse. The point closure standards in Table 3-1 must be met for the appropriate accuracy classification to be achieved. Table 5-1 lists general guidelines for the number of angle points between bearing checks and bearing discrepancies per angle point necessary to achieve the desired accuracy. A disadvantage of the traverse is the lack of field or computation checks on the accuracy until the traverse is connected to previously established traverse or triangulation networks of the appropriate order of accuracy.

Table 5-1
General Traverse Requirements for Horizontal Control Surveys

	USACE Classification	
Requirement	Second-Order	Third-Order
Number of angle points or distance between bearing checks	10 to 20 angle points	20 to 40 angle points but not > than 5 miles
Bearing discrepancy per angle point (not to exceed)	2.0 seconds	5.0 seconds

5-6. Various Forms of Traverse

Although there are various forms of traverses, they all have very basic and common methods of operation. The main differences between them is: the accuracy with which they are run, the different style of field notes required, and equipment used. Figure 5-1 shows some survey equipment used for traverse work. The field procedures are dictated by the type of traverse to be done. Even though there are equipment differences and level of accuracies achievable by the type of traverse to be done, the procedures for each can be generalized.

a. Control traverses. Control traverses typically are run for use in connection with all future surveys to be made in the area of consideration. They may be of First-, Second-, or Third-Order accuracy, depending on requirements.

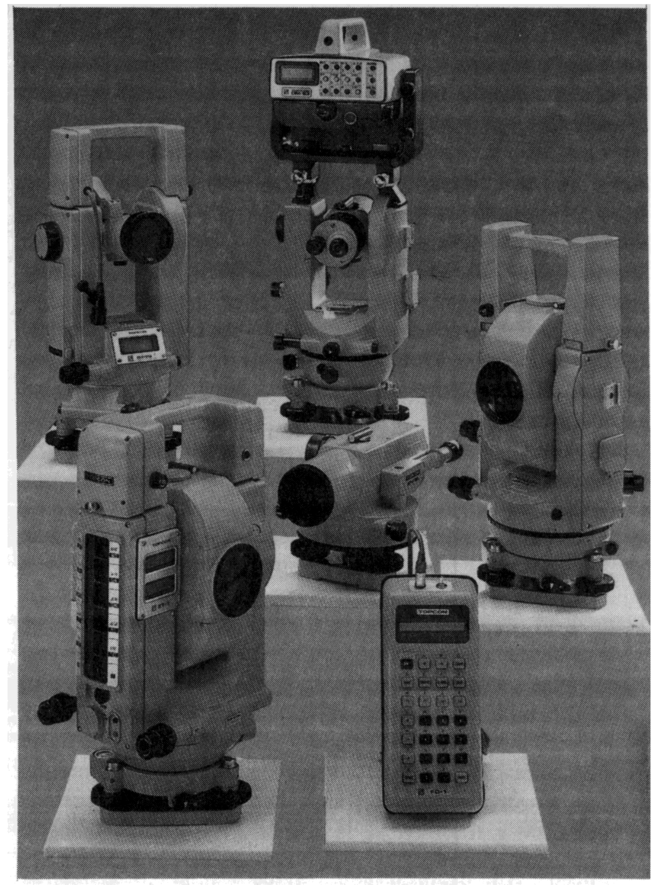


Figure 5-1. Typical survey equipment used in traverse and horizontal control work

(1) Most project requirements are satisfied with Second- or Third-Order accuracies. It is very unusual to have requirements in the USACE for First-Order traverse work. A Second-Order traverse requires special procedures both in the field and in the office to achieve the required accuracies for a particular project. A Second-Order traverse generally is necessary for special jobs or in areas where there is not sufficient triangulation control. Often, a secondary triangulation net is established instead of a traverse; but in flat areas, it is sometimes difficult to establish much control by triangulation because of sighting limitations, therefore it is advisable to run a Second-Order traverse, establishing permanent points at intervals of one mile or less.

(2) For a Second-Order traverse, the first step is to determine a good route. Start at a known point, preferably an NGS or USC&GS triangulation point. Plan the traverse to follow a route that will be centered as much on the work as possible, yet through areas that will not be bothered by construction, traffic, or other forms of congestion. The route should check into other known points as often as practicable. Depending on the application, examples of good traverse routes include ones along the edge of a road or the banks of a main river. After determining the route, it is best to next set permanent monuments (e.g., tee bars, brass caps, concrete, or some other suitable monument) at each angle point and any intermediate points desired. Refer to EM 1110-1-1002 for further guidance on survey markers and monumentation. Make sure there is a clear line of sight from angle point to angle point. Determine a numbering or naming system and mark all points at the time of setting.

(3) EDM or total station operation is less labor intensive, generally more productive, and provides as good as or better results than most conventional surveying methods. Manufacturer instructions for operation of the EDM or total station should be followed to determine distances between monuments and posts in the line.

(4) When using an EDM or total station, a minimum of two readings will be done before moving to the next post. All readings should agree within 0.001 foot with the original reading.

(5) Determination of angles should be done as soon as practicable after distance determination. Most total stations combine these two determinations (i.e., the distance and angle determinations). Special care should be taken with the type of sights used in the angle determination. Use as small a sight as possible given the requirements for clear vision of the target. The shorter the sight, the more important this consideration becomes. Fixed rigid sights should be used, not hand-held ones. If weather is especially hot or humid, early mornings or late evenings should be reserved for angle determination. Second-Order results generally do not require using lights or working at night, except under the most unusual of circumstances. It is desirable to use a directional theodolite or total station for all angle determinations. It is best to turn no less than three sets of angles, direct to right, if possible. Adequate results can be obtained with fewer angles if the best of equipment is used, but a few extra angles takes very little time compared with setting up instruments, traveling to the job, etc.. If results look slightly weak after turning the minimum angles, add extra angles, and consider closing to the horizon. All angle notes should be completely reduced and mean angle figured in the field and recorded along with the sketch. All adjustments should be made in the office. At the time of angle determinations, a sketch of the monument location should be made and a detailed description on how to find it written. This information will be used for making subsequent record of the survey monument.

b. Right-of-way traverse. A right-of-way traverse typically is a Third-Order traverse, starting and ending on known points. This type of traverse is usually run with a transit and 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 are 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 law suits 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. Be sure before getting to the field to know where to expect to find the corners. This information generally is available from recorded maps, the word of property owners, etc.

c. Stadia traverse. This type of traverse is not used for day-to-day operations to establish horizontal control. A stadia traverse can be used for establishment of rough (Third-Order or lower) horizontal control. 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. A stadia traverse typically is run along a route that will best suit the needs of the survey. The stadia points or stadia angle points will be set at locations that will best fulfill the purpose of the traverse and if possible will be set in protected locations for future use.

d. Topography traverse. This type of traverse is usually Third-Order or close to it and can be used for horizontal control in some instances. Its intended purpose is for controlling the mapping of an area previously determined. Because of this, it usually is necessary to determine elevations over the line prior to mapping. Some surveyors prefer to carry levels along with a transit survey, but this generally does not always provide results accurate enough for project requirements. Refer to EM 1110-1-1005 for guidance on topographic surveying.

e. Deflection traverse. A deflection traverse is used to indicate the way in which angles are turned with the transit or other instrument. This term usually applies to

Third-Order traverses with the only exception being compass traverses. A deflection angle is an angle that is measured right or left of the back tangent produced ahead of the instrument. The angle is measured by sighting the back tangent and plunging the instrument before turning to forward tangent. Accuracy is obtained by repeating this procedure using standard operating procedures for transit work. This method is especially good when office computations use a bearing and not an azimuth system.

f. Compass traverse. This type of traverse is exactly what its name implies: it is a traverse where the direction of the line is measured by a compass. In a compass traverse, no angles are turned. Distances are usually measured by stadia or paced if work is very rough. When using a compass to do a traverse, pacing is all it takes to determine distances comparable to the accuracies offered by the compass.

g. Azimuth traverse. Like a bearing, an azimuth denotes direction. Bearings break the directions into four quadrants, while an azimuth measures direction from one point. This point can be true North, South, or on some other base. Azimuth angles should always be turned to the right. This turning practice is particularly important for many directional instruments only read or turn to the right (e.g., most Wild instruments).

5-7. Traverse Classifications and Specifications

Traverses can be classified according to the type of instruments used to do them. All horizontal control surveys conducted by traverse will be classified based on the horizontal closure standards given in Table 3-1. Table 5-2 lists specific traverse requirements necessary to meet Second- and Third-Order type accuracies.

a. Second-Order. Second-Order traverse is used extensively for subdividing an area between First- and Second-Order triangulation and First-Order traverse. Second-Order traverse must originate and terminate on existing first or second order control that has been previously adjusted.

b. Third-Order. Third-Order traverse is normally used for detailed topographic mapping. Third-Order traverse must start and close on existing control stations of Third- or higher order accuracy.

c. Lower order. Traverses of lower than Third-Order are used for controlling points when a relatively large error in position is permissible. However, with the advent of new equipment, very little is saved either in time or money by using lower order traverse. The use of Third-Order methods should be carefully considered, even though the points are not to be monumented permanently. The map compilation requirement usually is that a horizontal control picture point for 1:50,000 mapping shall be located to within 6 m of its true relationship to the basic control. For 1:25,000 mapping the requirement is usually to within 3 m. The allowable errors permit accuracies to vary from generally 1 part in 500 to 1 part in 5,000, depending on the distance the lower order traverse must travel, the type of control at the start of the traverse, the desired accuracy of the control point and the methods and equipment used in the traverse.

Table 5-2
Traverse Requirements

Requirement	Second-Order	Third-Order
	Horizontal Directions or Angles	
Instrument	0.2" or 1.0"	1.0"
Number of observations	6 - 8	2 - 4
Rejection limit from mean	4 or 5"	5"
	Number of Azimuth	
Courses between azimuth checks not to exceed for:		
Tape	25	35 - 50
Electronic	12 - 16	25
	Azimuth Closure	
Probable error	2.0"	5.0"
	Azimuth Closure at	
Azimuth checkpoint not to exceed	10"*$N^{0.5}$ or 3" per station	15"*$N^{0.5}$ or 5" per station

Note: *N* is the number of stations carrying azimuth

Note: For Second-Order, the use of the traverse will dictate the type. For Third-Order, an astronomic azimuth station is sufficient. When two expressions are given, the formula which gives the smallest permissible value should be used.

5-8. Triangulation and Trilateration

Triangulation and trilateration may be used to establish horizontal control in areas where it is not practical to use other methods. Unless otherwise directed, horizontal control established by triangulation and trilateration will be at least Third-Order accuracy for most applications. As a general policy, the following minimum guidelines should be followed when performing triangulation and/or trilateration procedures:

a. Origin. When practicable, all triangulation and trilateration nets will originate from and tie in to existing control of equal or higher accuracy than the work to be performed. An exception to this policy would be when performing a triangulation or trilateration across a river or some obstacle as part of a chained traverse. In this case, a local baseline should be set.

b. Accuracy. The point closure standards in Table 3-1 must be met for the appropriate accuracy classification to be achieved. If project requirements are First-Order, refer to the FGCC/FCCS Standards and Specifications for Geodetic Control Networks (FGCS 1984).

c. Triangulation and trilateration methods. Triangulation and trilateration methods are similar in basic principle. A triangulation system consists of a series of joined or overlapping triangles in which an occasional line is measured and the balance of the sides are calculated from angles measured at the vertices of the triangle. A trilateration system also consists of a series of joined or overlapped triangles, but contrary to a triangulated system, a trilaterated system measures the lengths of all the sides of the triangles and only enough angles and directions to establish azimuth. Unfortunately, the nature of trilateration is such that only procedural guides are available to the field parties; they can only adhere to project instructions and specifics which should permit later computations to show the desired accuracy.

5-9. Bearing and Azimuth Determination

Bearing and azimuth determination are typically done as the basis for horizontal control surveys.

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 which the line makes with the meridian in that quadrant. The reference meridian may be either "true," magnetic, or assumed, while the bearings likewise also are "true," magnetic, or assumed.

(1) Observed bearings are those for which the actual bearing angles are directly obtained by survey field work, while calculated bearings are those for which the bearing angles are indirectly obtained by calculations.

(2) A true bearing is one whose reference meridian is a true meridian of the earth -- a meridian passing through the earth's poles and the projection of which passes through the celestial pole. The earth's poles are the geographic north and south poles. The celestial pole is geodetic north, the position of which is defined by its angular relationship to the North Star or Polaris. For general surveying purposes, it is assumed that for any given point on earth, true meridian is always the same, ensuring that any directions referred to the true meridian will remain the same regardless of time.

(3) A magnetic bearing is one whose reference meridian is the direction taken by a freely suspended magnetic needle (i.e., a compass). The magnetic poles are at some distance from the true poles, so the magnetic meridian typically is not parallel to the true meridian. The location of the magnetic poles is constantly changing, therefore the magnetic bearing between two points is not constant over time. Thus, magnetic bearings should only be used for reconnaissance work, rough surveys, and real-estate surveys.

(4) The angle between a true meridian and a magnetic meridian at the same point is called its magnetic declination. A line on the earth's surface which has the same magnetic declination throughout its length is called an "isogonic" line, while the line where magnetic north and a true meridian coincide is called an "agonic" line.

(5) An assumed bearing is a bearing whose prime meridian is assumed. In some cases, the relationship between an assumed bearing and the true meridian is given a definite calculable relationship. This is the case with most state plane grid coordinate systems used to make maps.

b. Bearing determination guidelines. All bearings used for engineering applications will be described by degrees, minutes, and seconds in the direction in which the line is progressing. The accuracy of its calculation is dependent on the exact measurements of distances and bearings. Also, the bearing will state first 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˚ (i.e., the bearing value always will be between 0˚ and 90˚).

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. Just like bearings, azimuths can be either "true," magnetic, or assumed, depending on the meridian to which they are referenced. Azimuths can be indicated from either the south point or the north point of a meridian.

(1) Assumed azimuths are often used for making maps and performing traverses. Typically, for USACE maps and traverse work, the angles (i.e., the azimuths) are determined in a clockwise direction from an assumed meridian. Assumed azimuths are sometimes referred to as “localized grid azimuths.”

(2) Azimuths, just like bearings, can be either observed or calculated. An observed azimuth is simply one which is read by using electronic instrumentation, transit, or compass in the field. Calculated azimuths are those obtained by computation. For example, a calculated azimuth may consist of adding to or subtracting field observed angles from known bearings or azimuths to determine other bearings or azimuths.

d. Azimuth determination guidelines. An azimuth will be determined as a line with a clockwise angle from the north or south end of a true or assumed meridian. The accuracy of its calculation is dependent on the exact measurement of distances and bearings. For traverse work using angle points, the requirements in Table 5-1 will be followed.

5-10. Astronomic Observation Requirements

a. 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. The specified intervals and the maximum deflection of the traverse will depend upon the accuracy of the traverse. The observation of an astronomic azimuth can be made by the hour angle method or altitude method. The method used will depend upon your location on earth and/or the order of accuracy required. A set of positions for an azimuth observation consists of half the required positions being observed using the rear station as an initial and half using the forward station as an initial. Using rear station, turn clockwise to forward station then to star, reverse telescope on star, then forward station and back to rear station. 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.

b. Position. For Second-Order traverse the observation of position for a Laplace azimuth will depend upon the use of the traverse, and the project instructions typically will specify when an astronomic position is required. For Second- and Third-Order, it is often necessary to observe astronomic positions to obtain the starting and terminating azimuth data.

5-11. Three-Point Resection

Three-point resection is a form of triangulation. Three-point resection will 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 will 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:

a. Location. Points for observation should be selected so as to give strong geometric figures.

b. Observation. If it is possible to sight more than three control points, the extra points should be included in the figure so that the computed position can be verified.

(1) When it is possible to occupy one of the control stations, it should be done as this added information will serve as a check on the computations and increase the accuracy because its position can also then be computed directly and checked by three-point computation. Occupation of a control station is especially important as it serves as a control of the bearing or direction of a line if a traverse were to originate from this same point.

(2) The interior and exterior angles shall be observed and recorded. The sum of these angles shall not vary by more than 3 seconds per angle from 360 degrees.

(3) Each angle will be turned not less than 2 times, but preferably 4 times, 2 direct and 2 inverted. Turn all angles to the right.

5-12. GPS Surveying

Establishing or densifying horizontal control with differential carrier-phase based GPS is often cost-effective, faster, more accurate, and more reliable than most conventional methods. The quality control statistics and large number of redundant measurements in GPS networks help to ensure viable results. Differential carrier-phase based GPS is particularly attractive for horizontal control surveying as compared with conventional surveys because intervisibility is not required between adjacent stations and GPS equipment is not limited by optics for its range of operations as are most conventional survey instruments. Horizontal control established with differential carrier-phase based GPS (in accordance with guidelines detailed in EM 1110-1-1003) will be of sufficient quality for most applications.

Chapter 6
Vertical Control Survey Techniques

6-1. General

This chapter outlines procedures to follow when performing conventional vertical control survey work. The material presented is not meant to be rigid methodology that cannot be tailored for a particular application. The mandatory part is the classifications, standards, style of keeping notes, and general procedures required to successfully complete the particular survey. Refer to EM 1110-1-1003 for GPS survey techniques for vertical control purposes as well as any of the applicable references listed in Appendix A for further guidance on information not covered in this chapter.

6-2. Vertical Control Surveys

a. The purpose of vertical control survey operations is to establish points at convenient intervals over the area to be surveyed. These established points can then serve as points of departure and closure for leveling operations and as reference marks during subsequent construction work. The NGS, National Ocean Service (NOS), United States Geological Survey (USGS), other Federal agencies, and many FOA have established control nets throughout the CONUS. Unless otherwise directed, these benchmarks will be used as a basis for all vertical control surveys. Usually NGS level circuits are found along main highways or railroads, while most FOA-maintained control is limited to project or survey areas. Exact locations of benchmark data and their values can be found in publications issued by the agency maintaining the circuit. Information on USACE-maintained points can be found at the District or Division level offices, but are generally maintained on the NGVD 29 or NAVD 88 datum. Level circuits maintained by other agencies are not necessarily on the same datum. Therefore, the surveyor must investigate what datum is being used in a particular level circuit before crossing into it. This will minimize the possibility of errors in subsequent calculations and operations.

b. Vertical control surveys can be done at First-, Second-, Third-, or Fourth-Order, although most applications can be satisfied with Third- or Fourth-Order vertical level work. Although there are various forms of vertical control surveys, all have similar methods of operation. The main differences between them are: the accuracy with which they are run, the slightly different style of field notes required, and the equipment used.

6-3. Direct Leveling

Direct leveling is usually referred to as differential leveling or spirit leveling. The actual differences in elevation are measured. In this method, a horizontal line of sight is established by using a sensitive level bubble in a level vial. The instrument is leveled and the line of sight of the instrument describes a horizontal plane. The difference in elevation between a known elevation and the height of instrument (HI) is determined. Next, the difference in elevation from the HI to an unknown point is derived by measuring the vertical distance with a precise or semiprecise level and leveling rods.

6-4. Indirect Leveling

Indirect leveling is a subdivision of leveling which does not measure the difference in elevation between points; instead other methods are used to determine elevation differences. There are two common methods of doing indirect leveling: trigonometric and barometric.

a. Trigonometric. This method applies the fundamentals of trigonometry to determine the differences in elevation by observing a horizontal distance and vertical angles above or below a horizontal plane to compute the vertical distance between points. Trigonometric leveling is generally used for Second-Order or lower order accuracies. Trigonometric leveling is especially effective in establishing control for profile lines, for strip photography, and in areas where the landscape is steep.

(1) In typical trigonometric leveling operations, it is only necessary to measure the HI, determine slope distance, read the vertical angle, and observe rod intercept. From these data, the vertical difference in elevation can be computed using the sine of the vertical angle and applying the rod difference as shown in Figure 6-1.

(2) Refinements to this technique that can reduce the chance for errors include doubling vertical angles, taking differences both ahead and back, and using mean values.

(3) If the horizontal distance is known between the instrument and the rod, it is not necessary to determine the slope distance. This is the case because the tangent of the vertical angle times the horizontal distance will give you the same answer as the sine of the angle times the slope distance.

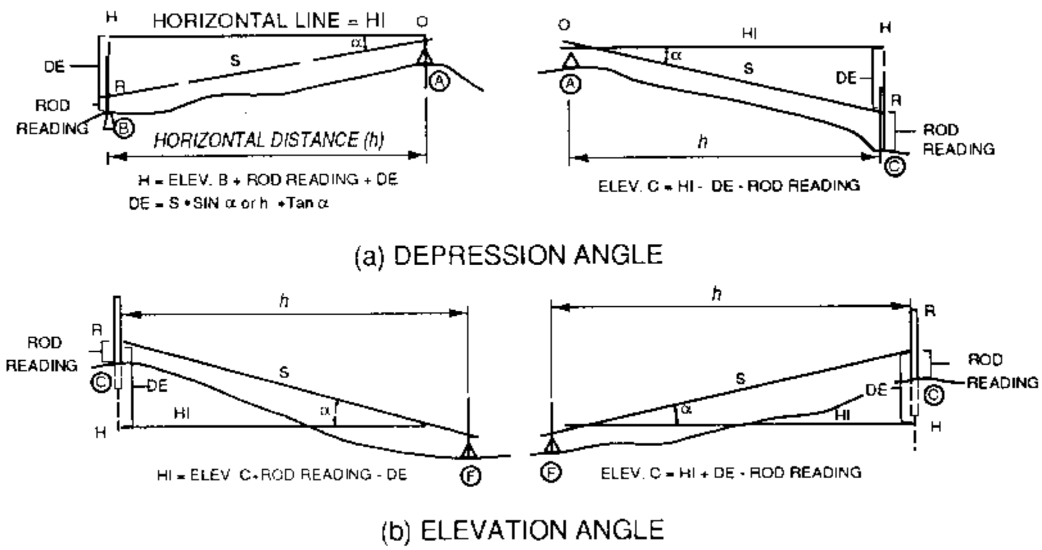


Figure 6-1. Trigonometric (or indirect) leveling

(4) Equipment. There are many types of instruments that may be used such as theodolites, transits, or alidades. The instrument used will depend upon the accuracy requirements.

(a) Theodolite. The instrument most commonly used is the 1-second directional theodolite.

(b) Transit. The transit is very seldom used by topographic surveyors today. The construction surveyor, however, will frequently be called on to use it. The procedure is the same as that of the theodolite.

(c) Alidade. With normal operating care, a telescopic alidade equipped with a Beaman arc can be used for lower order control leveling.

(d) Total station. An electronic-based instrument with features comparable to or better than those of a theodolite can be used. Manufacturers' procedures should be followed to achieve the point closure standards shown in Table 3-2.

(e) Digital levels. Digital (or Bar-Code) levels are used for fully automatic leveling. These instruments automatically measure, store, and compute heights and are capable of achieving Second-Order or higher accuracies. Manufacturers' procedures should be followed to achieve the point closure standards shown in Table 3-2.

(f) Semiprecise rods. When leveling in remote areas where the density of basic vertical control is scarce the semiprecise rod is generally used. The semiprecise rod should be graduated on the face to centimeters and on the back to half-foot intervals.

(g) Standard rods. 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. The Philadelphia rod graduated to hundredths of a foot can be used. Other rods that are graduated to centimeters can be used. Both types of rods are furnished with targets and verniers which 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 trigonometric level lines.

(h) Stadia rods. The standard stadia rods can also be used for lower order level lines. The stadia rod is graduated to the nearest 0.05 of a foot or 2 centimeters. These rods are generally equipped with targets or verniers, but if project specifications require, they can be estimated to hundredths of a foot.

b. Barometric. This method uses the differences in atmospheric pressure as observed with a barometer or altimeter to determine the differences in elevation between points. This is the least used method in surveying and the least accurate method of determining elevations. This method should only be used when one of the other methods is unfeasible or would involve great expense. Generally, this method is used for elevations when the map scale is to be 1:250,000 or smaller.

6-5. Reciprocal Leveling

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 6-2). Most level operations require a sight 300 or 400 feet long, and it's often possible in such operations to set backsights a similar distance away. Sometimes though it may be necessary to shoot 500 feet, 1,000 feet, or even further in order to span across a river, canyon, or other obstacle. Where such spans must be leveled across, reciprocal leveling is appropriate.

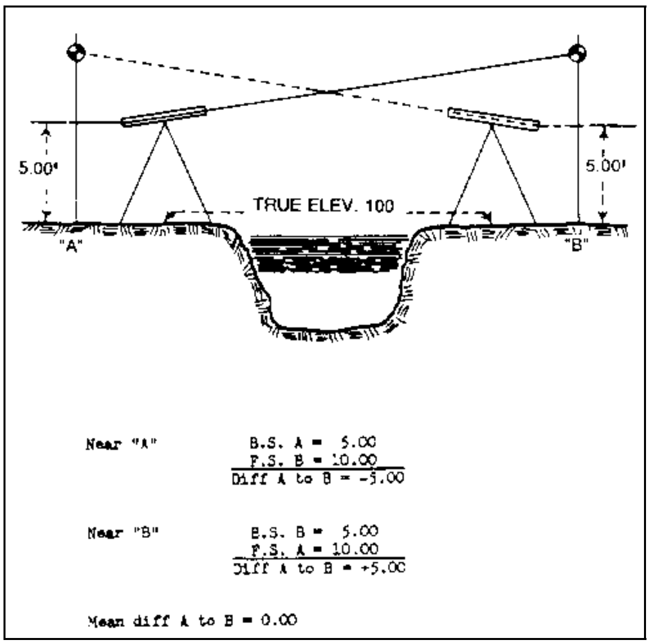


Figure 6-2. Reciprocal leveling

a. Procedure. Assume point "A" and "B" are turns on opposite sides of the obstacle to be spanned (Figure 6-2). Points A and B are assumed to be able to be sighted from an instrument set up opposite to it. Two rods should be used, one at point A and the other at point B. The rods should be first compared on a common turn to be sure they will read exactly the same (i.e., they are calibrated or a matched pair). 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 instrument around to point B, read B, then turn to read A and again determine the difference in elevation. Example calculations are shown in Figure 6-2. The mean of the two differences is the true difference to be applied to the elevation of A to get a true value of B. If the long sight is difficult to determine, it is suggested that several tries be made and an average determined. For more precise results, it may be necessary to take a series of 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 recenter the level bubble (if applicable) and reset the target after each observation.

b. Cautions. Reciprocal leveling assumes the conditions during the survey do not change significantly for the two positions of the level. Two factors that typically may affect the result when the lengths of the sights are long are unequal expansion of the parts of the instrument and variations in atmospheric refraction. To minimize these effects, it is recommended observations be made on a cloudy, cool day if possible. Protecting the instrument with an umbrella and rapid careful collection of the readings also help to minimize these two factors. Reciprocal leveling with two instruments should never be done unless both instruments are used on both sides 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 are of similar power and sensitivity.

6-6. Three-Wire Leveling

a. Accuracy. Three-wire leveling can be used to achieve any level of accuracy from First- to Third-Order, depending on the care in which the work is conducted. However, most applications do not require the accuracies possible with three-wire leveling and due to its labor-intensive nature, sound judgment must be exercised when deciding whether or not three-wire leveling should be used to establish vertical control for a project.

b. Methodology. Three-wire leveling can be applied whenever the reticule of the level has stadia lines and stub-stadia that are spaced so that the stadia intercept is 0.3 foot at 100 feet, rather than the more typical 1.0 foot at 100 feet. The stub-stadia 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. A Zeiss Ni2 Self-Leveling level is an example of a typical instrument designed specifically for three-wire leveling. In three-wire leveling, 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. For example, in a typical reading, 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 foot -- 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 which are designed for ordinary leveling.

6-7. 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. Halfway between benchmarks, the rods should be interchanged to minimize the possible effects of index error. Two rods are recommended when using a self-leveling level, as this takes full advantage of the productivity possible with this type of instrument.

6-8. Tidal Benchmarks and Datums

Refer to EM 1110-2-1003 and FM 5-441 for guidance on the establishment of tidal benchmarks and datums.

6-9. Leveling Operations

a. General. The leveling operation consists of holding a rod vertically on a point of known elevation. A level reading is then made through the telescope on the rod, known as a backsight or 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 or HI, is determined. Another rod is placed on a point of unknown elevation, and a foresight or FS reading is taken. By subtracting the FS reading from the HI, the elevation of the new point is established. After the FS 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 BS for a new HI and the sighting on the front rod is an FS for a new elevation. The points on which the rods are held for FSs and BSs are called turning points. Other FSs 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.

b. Second-Order leveling operations. Leveling of Second-Order accuracy levels is usually limited to control of a large area or on jobs where grades are critical. An example where Second-Order accuracy may be required is a canal where a concrete lining is used and the grades are exceptionally flat. Another example is a tunnel being dug underground for several miles where the ends may be miles apart by the route the control levels are to be run, but the aboveground distance between the ends is relatively short. On some jobs requiring such high levels of accuracy, it is desirable to close the loop being run back to the point of beginning because closing into another existing benchmark of equal or higher order may be impractical, especially if the check-in benchmark is in another leveling circuit from a completely different area.

c. Second-Order leveling accuracy. Second-Order leveling point closure standards for vertical control surveys are shown in Table 3-2. Second-Order leveling consists of lines run in only one direction, 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.

d. Second-Order leveling equipment. The type of equipment needed is dependent on the accuracy requirements.

(1) Second-Order level. The instrument used in Second-Order leveling can be a total station, level, or equivalent that provides precise leveling and is constructed of materials having a low coefficient of thermal expansion. The instrument must employ the normal tilting device, and the design of the instrument must allow the bubble in the level vial to be observed by the instrument man without moving from his normal observing position. If the instrument is a level, it must be of the Dumpy type, incorporating a three-wire reticule, an

inverting eyepiece, and a three-screw base. Generally a graduated tilting screw 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. These factors may vary individually, but when one factor is weakened, another factor must be strengthened to maintain the specified order of accuracy. Instruments are rated and tested according to their ability to maintain the specified order of accuracy. Only those rated as precise geodetic quality instruments (such as Wild N-3) may be used for Second-Order work.

(2) Second-Order rods. Precise level rods are required when running Second-Order levels. There are many precise rods, and any of them may be used as long as they are equivalent in accuracy to the rod described. The rods must be of the one piece, invar strip type, with the least graduation on the invar strip of 1 cm. The invar strip is 25 mm wide and 1 mm thick and is mounted in a shallow groove in a single piece of well-seasoned wood. The groove is slightly wider than the strip and deep enough so that the strip is free to move. The rod has a metal footpiece to which the strip is securely fastened. In order to eliminate any error due to sagging of the invar strip when the rod is held erect, it is placed under tension by a stiff spring, set into a recess in the top of the staff and bearing against a small brass angle plate attached to the top of the strip. This tension spring also allows the strip to expand or contract because of temperature changes. The front of the rod is graduated in meters, decimeters, and centimeters on the invar strip. The back of the rod must be graduated in feet and tenths of feet, or yards and tenths of yards. These rods must be standardized by the National Institute of Standards and Technology and their index and length corrections determined. The 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 which may affect their accuracy. The precise level rod is a scientific instrument and must be treated as such, not only when in use but during storage and transporting. When in use the rods should be alternately carried by the handles and face up on the shoulder. If the rods are always carried with the face down or up, they will become slightly bowed and the reading will be too large. Therefore, the rods should be carried an equal amount of time by the handle and face up on the shoulder. 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.

(3) Instrument test 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 instrument should be carefully determined. This stadia factor is required in the computation of the length from the stadia intervals and in computing the allowable error. This determination must be done for each level used in the field. Should new cross wires be installed in the field, a redetermination of the constant must be made before using the level in the survey. The observations and computations for this determination must be recorded as a permanent record along with the time and date and kept with the project files. The determination is made by comparing the stadia intervals observed over a known distance course. The course should be laid out on a reasonably level track, roadway, or sidewalk, and nails or other marks placed in a straight line at measured distances of 0, 25, 35, 45, 55, 65, and 75 m. The center of the instrument and the zero point of the instrument may not be the same for the instrument being used. The Wild N-3 level has an additive constant of -20 cm. When determining the stadia constant, therefore, the instrument must be plumbed 20 cm forward of the zero stake. For other instruments, the manufacturer's manual should be consulted. Read the rod at each of the six points and record the intervals. The level bubble need not be accurately centered, but should at least be free of the ends of the tube. The half-wire intervals should be computed as a check against erroneous readings. The sum of the total intervals for the six readings should be computed. The stadia constant is the sum of these measured distances (300 m) divided by the sum of the six total wire intervals. As a check against gross errors each separate observation should be computed. The average of the six separate computations serves as a numerical check on the computations. Any tendency for the six computed values to creep in one direction will be good evidence that some error in the measurement of the distance has been made from the center of the instrument to the zero point and then to the first of the six rod points.

(b) Determination of "C" factor. Each day, just before the leveling is begun or immediately after the beginning of the day's observations, and immediately following any instance when the level is subjected to unusual shock, the error of the level or C factor must be determined. This determination may be done during the regular course of leveling or over a special 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 6-3):

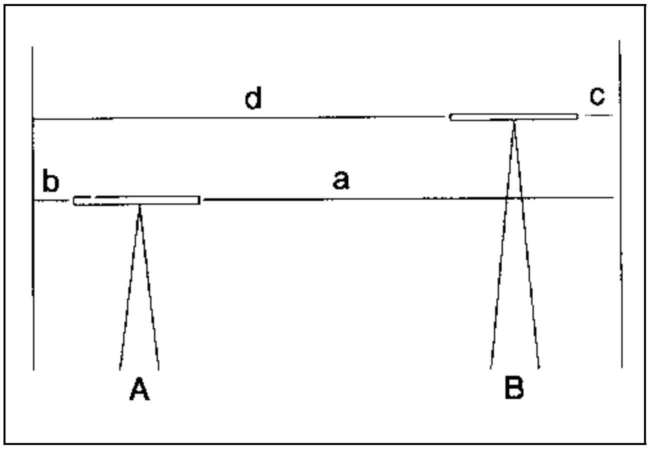


Figure 6-3. Procedure for C factor

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 back rodman and have the rod placed about 10 m back from the instrument; read the rod "b," over the instrument to a position "B" about 10 m 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 basically the ratio of the required rod-reading correction to the corresponding subtended interval, or:

$$C = \frac{(\Sigma \textit{ near rod readings}) - (\Sigma \textit{ distant rod readings})}{(\Sigma \textit{ distant rod readings}) - (\Sigma \textit{ near rod readings})}$$

The total correction for curvature and refraction must be applied to each distant rod reading before using them in the above equation. 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 at once if the C factor is greater than 0.004 for instruments with a stadia constant of approximately 1/100, 0.007 for stadia constants of 1/200, and 0.010 for stadia constants of 1/333. It is desirable to have the determination of the C factor made under the anticipated conditions 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 essential in computing the level corrections when making office computations for that particular day's work.

(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 leveling rods. Precise level rods must be tested once each week 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 m on a 3-m length of rod, the rod level must be adjusted. This 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.

e. Second-Order leveling monumentation. All benchmarks used to monument on Second-Order level lines will conform to criteria in EM 1110-1-1002. In general, all benchmarks used to monument Second-Order level lines shall be standard USACE brass caps set in concrete. The concrete should be placed in holes deep enough to avoid local disturbance, such as spading, soft-surface heaving due to winter rains, frost action, etc. If the brass cap is not attached to an iron pipe or tip, use some type of metal to reinforce the concrete prior to embedding the brass cap. Concrete should be placed in a position that is protected as much as possible. If possible, benchmarks should be set close to a fence line, yet far enough away to permit plumbing of level rod. Do not set the monument closer than 4 feet to a fence post as when the post is replaced, the benchmark usually will be disturbed. If practical, when a benchmark is monumented near a fence, paint the top 18 inches of the nearest fence post white to aid in identification of the location of the benchmark. Each brass cap must be stamped to identify it by the adopted method detailed in EM 1110-1-1002. In addition to stamping a local number or name on the cap, it is desirable to stamp true elevation on the brass cap after final 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 done after monument construction.

f. Second-Order leveling notes. Notes for Second-Order levels will be kept in a manner approved by the immediate supervisor. A set style cannot be developed due to different types of equipment that can be employed. Elevations will not be carried in the field as they will be adjusted by the field office and closures approved prior to marking benches.

g. Third-Order or construction layout leveling. All levels run for traverse profiles, temporary benchmarks, control of cross-sections, slope stakes, soundings, topographic mapping, structure layout and miscellaneous construction layout and miscellaneous construction staking shall be Third-Order or construction levels as detailed in Table 3-2, unless otherwise directed. All levels will originate from and tie into existing control. No level line shall be stubbed off or dead ended unless by specific instructions by a designated immediate supervisor or written directive.

h. Third-Order and lower leveling accuracy. All accuracy requirements for USACE vertical control surveys will conform to the point closure standards shown in Table 3-2. The required accuracy for Third-Order levels is 0.050 M feet where M is the length of the level line in miles, while 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, although this is not a preferred method. Direct measure of the line is the preferred method.

i. Third-Order and lower leveling equipment. The type of equipment needed is dependent upon the accuracy requirements.

(1) Third-Order level. A semiprecise level should be used for Third-Order levels, such as the tilting Dumpy type, three-wire reticule, or equivalent.

(2) Third-Order rods. The rods should be graduated in feet, tenths, and hundredths of feet. The Philadelphia rod or its equivalent is acceptable. However, the project specifications will sometimes require that semiprecise 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 semiprecise rods.

(3) Lower order. The type of spirit level instrument used should be a type that ensures an accuracy in keeping with required control point accuracy. The precise type level should not be used on lower order work when other less precise instruments are available. The Fennel tilting level, Dumpy level, Wye level, or their equivalent are examples of levels that can be used. The rods should be adequate for obtaining the required accuracy. The precise invar strip rods should not be used. Any stadia rod with least readings of five-hundredths of a foot or 1 centimeter will be satisfactory. The use of turning pins and/or plates will depend upon the type of terrain. Terrain permitting, the rods may be placed on reasonably firm stones or roadways.

j. Third-Order and lower leveling monumentation. The level line shall be tied to all existing benchmarks along or adjacent to the line 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 necessitate monuments be placed more often.

(1) It is desirable to set benches on permanent structures. Examples of permanent structures include head walls, bridge abutments, pipes, etc. Large spikes driven into tree roots, telephone poles at the base, and fence posts generally are acceptable for this level of work. All temporary benchmarks must have full description of what they are and where they are located. Unless they are on a turn, they may not be considered to be temporary benchmarks. No closures shown by an intermediate shot will be accepted. All temporary benchmarks must have a name or number for future identification.

(2) Turning points are a very important part of leveling. The rodman will select a suitable point or drive a turning pin in the ground until rigid with no possibility of movement. All 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. If solid rocks are being used for turns they must be marked with crayon or paint prior to taking reading. Be sure the rod will spin free on the high point.

(3) 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.

k. Third-Order and lower leveling notes. Complete notations or sketches will be made to identify shots. All

Third-Order or lower level notes will 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 an immediate supervisor or written directive. Any change in rod reading shall be initialed and dated so there is no doubt as to when correction was made. Cross out erroneous shots -- never erase them. The instrument man shall take care to keep peg notes on all turns in the standard field book. The notes will be dated and noted as to what line is being run, station occupied, identification of turns, etc.

6-10. GPS Surveying

Establishing or densification of vertical control with differential carrier-phase based GPS is often difficult. GPS provides height (h) or height difference (h) in terms of height above or below the WGS 84 reference ellipsoid. These ellipsoid heights (h) are not equivalent to orthometric heights (elevations), which would be obtained from conventional differential leveling. Therefore, users of GPS must exercise extreme caution in applying GPS height determinations to projects which are based on conventional orthometric elevations. Using current methodology (i.e., geoid modeling and transformation software), the best accuracy achievable is Third-Order. Therefore, the use of differential carrier-phase based GPS techniques for establishing or densifying of vertical control should be limited to applications where this level of accuracy is acceptable. Refer to EM 1110-1-1003 for guidance concerning GPS vertical control surveys. Point closure standards shown in Table 3-2 should be used to classify subsequent results.

Chapter 7
Miscellaneous Information on Survey and Instrument Operations

7-1. General

The purpose of this chapter is to provide general survey operations guidance necessary to obtain the accuracy requirements of the USACE. The user should make special note of survey note requirements and the general principles of survey operation.

7-2. Field Notes

a. Field notes. All field notes will be recorded in a standard hardcover field book as the measurements are made in the field. The typical dimensions of such a field book are 4-7/8 inches by 7-1/2 inches.

(1) All field note entries shall be made with a black lead pencil or black ink. Notations made by other than the original surveyor shall be made with a colored pencil so a clear distinction exists between the field observations and subsequent corrections, adjustments, comments, or supplemental data.

(2) The first two pages of each field book shall be reserved for the book index and shall not be numbered. The index should contain the date and description of the survey. The description will indicate the type of field activity performed: traverse, levels, topography, etc. Also, the index should list the actual pages used in the field book for the particular description entry. The remainder of the field book shall contain the actual field data and shall be numbered beginning at page one.

(3) The first page of each entry should contain at the top left side of the page the name of the installation or project location and a specific project title (e.g., James River - Kingsmill Dam), the type of work being done, and a detailed description of the work. This description should include as much information as necessary to detail what was done during the course of the survey.

(4) At the top of the right side of the right half of the page, the actual date of the survey, weather conditions, type and serial number of instruments used, members of the crew and their assignment, map or field book references, and other remarks as necessary for a complete understanding of the survey shall be written.

(5) No data will be written on scratch paper and/or on the back page of the field book and copied into the book later. Also, details of the survey will not be carried in the mind of the surveyor or until the end of the job and then entered into the field book. Information copied or transferred into the field book or carried in the mind of the surveyor is subject to more error than data recorded as they are obtained. Systematic recording of data will help to improve accuracy. No erasures should be made in the field book. If errors are made, they will be crossed through and the correct ones will be written in such a way that the original data remain legible. No figure should ever be written over the top of another nor should any figure be erased. If a whole page is in error, the complete page will be lined or crossed through and the word "VOID" will be written in large letters diagonally across the page. A cross reference will be entered on the voided page showing the book and page number where the correct information may be found. Also, an explanation of the error and the correction will be entered in the field notes.

(6) If a data collector is used, only setup information (i.e., description, HI, sketch, etc.) and every 20th shot should be recorded in the field book. This information is used to check the instrument for systematic errors.

(7) When it is necessary to copy information from another field book or other source, a note will be made which clearly states that the information was copied and the source from which it came.

(8) If the notes are a continuation from another field book, a description will be written in the field book to the effect "NOTES CONTINUED FROM BK XXXX PAGE XX." A similar description (e.g., CONTINUED IN BOOK XXXX FROM PAGE XX) will be written on the last page of each section of notes if those notes are to be continued either in another book or on another page which is not adjacent to the current page.

(9) The sketch should show all the details, dimensions and explanatory notes required. The sketch should be written on a whole page whenever possible. If necessary, multiple pages with the sketch divided equally among the pages should be used if the sketch has too many details to be shown on one page. Sketches of structure sections must be well drawn as they are often the basis for working drawings of existing structure(s). If applicable, show the center-line station of all creeks and draws and edge of channel on all rivers and streams crossed.

(10) Note in the field book any conversations with owners regarding property corners. Use owner's full name if possible.

(11) At the end of each day of work, the field notes shall be signed and dated by the crew chief or individual responsible for the work.

b. Minimum horizontal control survey field note requirements.

(1) When using the appropriate applicable equipment (e.g., EDM, total stations, etc.) to do a traverse, the field notes shall contain the height of the instrument (i.e., HI) above the station occupied, the target height (i.e., TH) above the station being measured to, both the horizontal angles and the vertical angles, and distance readings obtained with the instrument.

(2) Even though the EDM, total station, etc. being used may be capable of computing and displaying a horizontal distance and a difference in elevation based on the slope distance and vertical angle obtained by the instrument, the vertical angles shall still be recorded in the field book. All these measurements shall be clearly labeled as vertical angle, slope distance, and horizontal distance.

(3) In addition to the measured angles, a description of the point occupied shall be included. This description shall include the type of monument (e.g., brass disk, RR spike, 1/2" re-bar, etc.), general location (e.g., 15' east of north end of Kingsmill Dam centerline & 20' south of fire hydrant), and type of material point is set in (e.g., chiseled cross on concrete slab, flush with gravel road, etc.). A sketch of the location of the point relative to existing physical features and reference ties shall be made and included in the notes.

(4) If a horizontal control line is used, a sketch of it shall be made and included in the notes. This sketch does not need to be drawn to scale, but it should include the relative position of one point to the next and the basic control used. Figure 7-1 shows some examples of field notes taken during a typical horizontal control survey.

c. Minimum vertical control survey field note requirements.

(1) A short description of the course of the level line shall be entered in the field book.

(2) Entries shall be made in the book that give the references to the traverse notes and other existing data used for the basic elevations (e.g., TRAVERSE BOOK XXXX PAGE XX, USGS Quad XXXXXX, NOS Chart XXXX, etc.).

(3) A complete description of each point on which an elevation is established shall be recorded in the field book adjacent to the station designation. Figure 7-2 shows some examples of field notes taken during a typical vertical control survey.

7-3. Entry Rights

When entering property to conduct a control survey, the rights of the property owner will be respected. The following details some minimum guidelines to follow in an effort to respect the rights of the property owner.

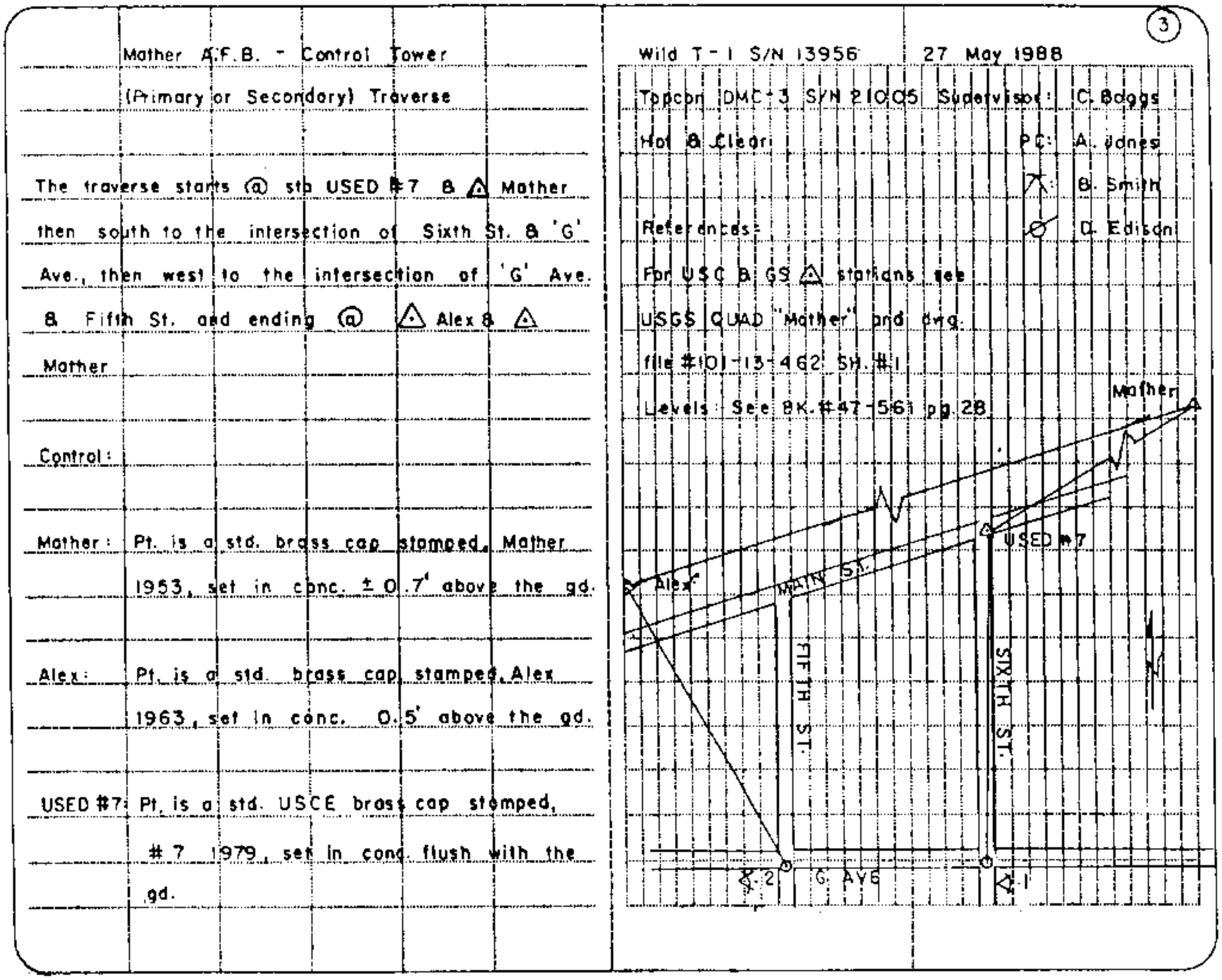
a. Permission to enter a military installation and other private property will always be acquired by the District prior to entering such property. While on the military installation, members of the survey crew will adhere to all of the stipulations (e.g., rules, regulations, directives, verbal guidance, etc.) set forth by the Base Installation Commander or his designated representative. The same basic guidelines are applicable when the right to enter private property is given.

b. Government and private property shall be protected at all times. The right to enter a property does not give the crew the license to destroy or cause excessive damage to the property. Every effort should be made not to damage or cut trees, shrubs, plants, etc. on the property. If such must be done, the Base Installation Commander or in the case of private property, the private property owner, is the only person who can grant permission to do so.

c. As practicable, the property entered shall be returned to its condition prior to entry once the survey is completed. Gates and other structures should be left in the position in which they were found prior to entry. If a gate is closed, do not leave it open for any long period of time.

d. Return all borrowed property (e.g., keys, maps, etc.) as instructed by the property owner or designated representative.

e. Survey points should be placed in such a way as to not obstruct the operations of the property owners or be offensive to their view. Monuments set as a result of the survey should be set below ground level to prevent damage by or to any equipment or vehicles. Extra care must be taken when setting a survey point at or near airports.



3

Mather A.F.B. - Control Tower

(Primary or Secondary) Traverse

The traverse starts @ stb USED #7 & △ Mather then south to the intersection of Sixth St. & 'G' Ave., then west to the intersection of 'G' Ave. & Fifth St. and ending @ △ Alex & △ Mather

Control:

Mather: Pt. is a std. brass cap stamped, Mather 1953, set in conc. ± 0.7' above the gd.

Alex: Pt. is a std. brass cap stamped, Alex 1963, set in conc. 0.5' above the gd.

USED #7: Pt. is a std. USCE brass cap stamped, # 7 1979, set in conc. flush with the gd.

Wild T-1 S/N 13956 | 27 May 1988

Topcon DMC-3 S/N 21005 Supervisor: C. Boggs

Hot & Clear

PC: A. Jones

⊼: B. Smith

⌀: D. Edison

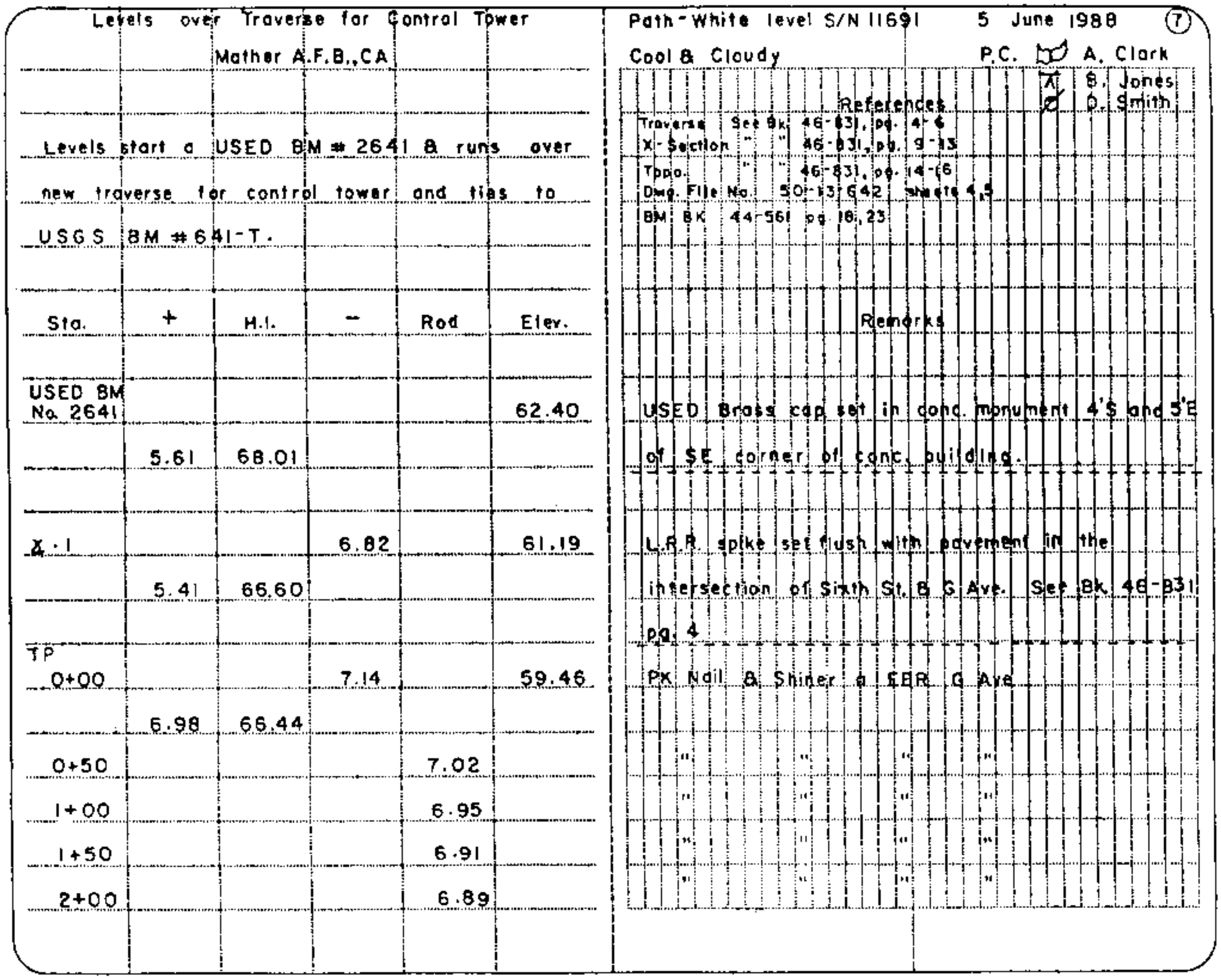
References:

For USC & GS △ stations see USGS QUAD "Mather" and dwg. file #101-13-462 SH. #1

Levels: See BK. #47-561 pg. 28

Figure 7-1. Field notes taken during a typical horizontal control survey

f. Any pre-marks set on military installations or private property will be removed as soon as possible after the survey work is completed or the Base Installation Commander, property owner, and/or designated representative requests such.



Levels over Traverse for Control Tower

Mather A.F.B., CA

Levels start a USED BM # 2641 & runs over new traverse for control tower and ties to USGS BM # 641-T.

Path-White level S/N 11691 5 June 1988 (7)

Cool & Cloudy

P.C. A. Clark

B. Jones

D. Smith

References

Traverse See Bk. 46-831, pg. 4-6

X-Section " " 46-831, pg. 9-13

Topo. " " 46-831, pg. 14-16

Dwg. File No. 50-13-642 sheets 4,5

BM Bk 44-561 pg 18, 23

Sta.	+	H.I.	−	Rod	Elev.	Remarks
USED BM No. 2641					62.40	USED Brass cap set in conc. monument 4'S and 5'E of SE corner of conc. building.
	5.61	68.01				
X-1			6.82		61.19	L.R.R. spike set flush with pavement in the intersection of Sixth St. & G Ave. See Bk. 46-831 pg. 4
	5.41	66.60				
TP 0+00			7.14		59.46	PK Nail & Shiner @ SER G Ave.
	6.98	66.44				
0+50				7.02		" " " "
1+00				6.95		" " " "
1+50				6.91		" " " "
2+00				6.89		" " " "

Figure 7-2. Field notes taken during a typical vertical control survey

Chapter 8
Survey Adjustments for Conventional Surveys

8-1. General

The adjustment of survey and photogrammetric data is a critical component in the determination of reliable coordinates, directions, distances, and elevation data. The adjustment technique and the analysis of the estimated parameters should be given the same priority as the data collection and recording procedures. An adjustment is a method of dealing with redundant data. Redundancy can be considered excess information. If the redundancy of a system is zero, the system of observations would not warrant an adjustment. For example, if a surveyor measured the distance from two known survey points to an unknown location (range - range intersection), the two distances from the known points would uniquely determine the two-dimensional location of the unknown point. In this example there is no extra or redundant data, therefore, an adjustment would not be warranted.

a. Survey computations, whether made on a local system or a standardized accepted system (e.g., SPCS), are for all intents and purposes identical. The adjustment of raw survey data is treated as independent observations and adjusted as part of a total network. A variety of methods may be used to adjust the survey data, including compass (or Bowditch) rule, transit rule, the Crandall Method, and the method of least squares. This chapter describes some of the methods used to perform horizontal and vertical adjustments and provides guidance in evaluating the adequacy and accuracy of the adjustment results.

b. Differential carrier phase GPS survey observations are adjusted no differently than conventional surveys. Each three-dimensional GPS baseline vector is treated as a separate distance observation and adjusted as part of a trilateration network. A variety of the techniques developed in this chapter can be used to adjust observed GPS baselines to fit existing control. However, they are usually adjusted by least squares. Refer to EM 1110-1-1003 for further guidance on GPS baseline adjustment.

8-2. Adjustment Methods

An adjustment may involve a mean of observations, balancing of a traverse, and an adjustment by least squares. This chapter will briefly address traverse balancing and the concept of a weighted mean. The main scope of the chapter will deal with the adjustment of data using the method of least squares. The method of least squares is the most prevalent adjustment technique utilized by commercial software packages.

8-3. Traverse Adjustment (Balancing)

The method of traverse adjustment is widely used by the land surveying and engineering communities. This method of adjustment is easy to perform and provides adequate results for many survey applications. The method of traverse adjustment depends on the precision of the directions as compared to the precision of the distances. The equations necessary for each adjustment method are available in any elementary survey text.

a. Crandall rule. The Crandall rule is used when the angular measurements (directions) 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. Also, with the advent of the personal computer, a traditional least squares adjustment can be performed with little effort.

b. Transit rule. The transit rule is utilized when the angular measurements are of greater precision than the linear measurements. For example, if a surveyor was using a transit or theodolite for angular measurements and stadia for linear measurements, the transit rule adjustment would be applicable. This method is rarely used because modern distance measuring equipment (DME) and electronic theodolites provide distance and angular measurements with equal precision.

c. Compass rule. The compass rule adjustment is used when the angular and linear measurements are of equal precision. This is the most widely used traverse adjustment method. Since the angular and linear precision are considered equivalent, the angular error is distributed equally throughout the traverse. For example, the sum of the interior angles of a five-sided traverse should equal 540^{o} 00' 00".0, but if the sum of the measured angles equals 540^{o} 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.

d. Example compass rule adjustment. A four-sided closed traverse was performed (Figure 8-1) using a twenty-second (20") theodolite and a one-hundred-foot (100') steel tape. The linear and angular measurements were considered of equal precision, therefore, the compass rule was utilized to adjust the traverse. The observed and adjusted angles, azimuths, latitudes, and departures are listed in Table 8-1. The following four steps demonstrate how a compass rule adjustment is performed for a loop traverse.

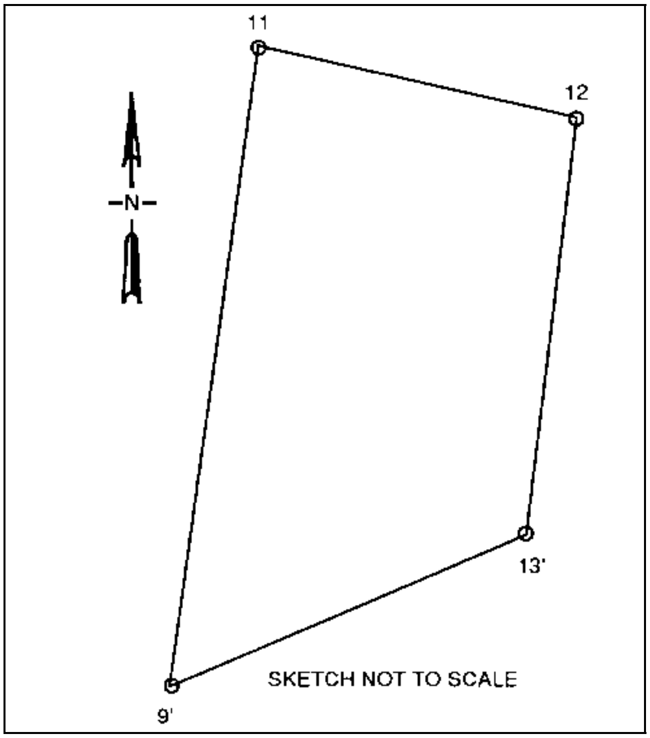


Figure 8-1. Loop traverse

Step 1. The angular error (α_e) of the polygon is computed by differencing the measured (α_m) and true (α_t) angular closure. The measured angular closure is the summation of the interior or exterior horizontal angles in the traverse. If interior angles were measured when performing a loop traverse, the true angular closure equals:

$$\alpha_t = (n-2) * 180^o$$

n = number of sides in the traverse

α_t = true angular closure

If exterior angles were measured when performing a loop traverse, the true (α_t) angular closure would equal:

$$\alpha_t = (n+2) * 180^o$$

n = number of sides in the traverse

α_t = true angular closure

Angular Error

$$\alpha_e = (\alpha_m - \alpha_t)$$

α_e = angular error

The angular error (α_e) is divided by the number of sides (n) in the traverse and is distributed equally to all of the measured angles. If the angular error is negative, the error is added to all the angles in the traverse. If the angular error is positive, the error is subtracted from all of the angles in the traverse. After the angular misclosure has been distributed throughout the traverse, the summation of the interior or exterior angles should equal the true angular closure (α_t). The four-sided closed traverse in Figure 8-1 and Table 8-1 has an angular error (α_e) of four arc seconds (4"). In this example, the angular error is negative; therefore, to balance the angles within the traverse, one arc second (1") must be added to each angle in the traverse.

Step 2. The horizontal angles are converted to bearings or azimuths. To compute the "true" bearing or azimuth of each line in the traverse, one known bearing or azimuth must be available prior to the adjustment process. If a known bearing or azimuth is not available, a "false" bearing can be used to perform the adjustment process. If a "false" bearing is utilized, the traverse will be oriented relative to this bearing. The example in Figure 8-1 and Table 8-1 has a known azimuth between stations 11 and 12. The azimuth for each line in the traverse was computed by adding the back azimuth between stations and the angle to the right. The back azimuth is computed by adding or subtracting one-hundred and eighty degrees (180°) from the forward azimuth. The angle right equals the measured interior angle.

Back Azimuth

If the forward azimuth $\alpha_{ab} < 180^o$

The backward azimuth $\alpha_{ba} = \alpha_{ab} + 180^o$

If the forward azimuth $\alpha_{ab} > 180^o$

The backward azimuth $\alpha_{ba} = \alpha_{ab} - 180^o$

Table 8-1
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			
		$\Sigma = 360°$									
$\Sigma\alpha_m =$	359°-59'-56"										
$\alpha_t =$	360°										
$\alpha_e =$	-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 $\left(\frac{P}{D}\right)$

Step 3. The latitude and departure for each course in the traverse are computed using the bearing or azimuth and distance of the line. If bearings are utilized, the latitude and departure must be identified as negative or positive. The latitude of a line increases south to north. The departure of a course increases from west to east. In the example from Figure 8-1, the line from station 13' to 12 would have a positive latitude and departure. The line from station 12 to 11 would have a positive latitude and negative departure. When azimuths are utilized, the algebraic sign (+/-) of the latitude and departure is accounted for in the trigonometric functions.

Latitude and Departure

$$\Delta N_m = \cos(BRG) * l_{ij}$$

$$\Delta E_m = \sin(BRG) * l_{ij}$$

or

$$\Delta N_m = \cos(\alpha) * l_{ij}$$

$$\Delta E_m = \sin(\alpha) * l_{ij}$$

where

ΔN_m = measured latitude

ΔE_m = measured departure

BRG = bearing

α = azimuth

l_{ij} = distance between stations i and j

Step 4. The summation of the measured latitudes ($\Sigma\Delta N_m$) and departures ($\Sigma\Delta E_m$) represents the error in northing and easting of the traverse. The northing and easting error is distributed throughout the traverse proportional to the distance of each line (l_{ij}) and the perimeter distance of the traverse (D). The precision of the traverse (P) is equal to the line of closure (ρ) divided by the perimeter distance of the traverse.

Latitude and Departure Error

$$\delta N_e = (l_{ij}/D) * \Sigma\Delta N_m$$

$$\delta E_e = (l_{ij}/D) * \Sigma\Delta E_m$$

Traverse Precision

$$P = \rho/D$$

where

P = traverse precision

ρ = $((\Sigma\Delta N)^2 + (\Sigma\Delta E)^2)^{0.5}$ (line of closure)

D = $\Sigma\, l_i$

$\Sigma\Delta N_m$ = summation of the measured northings within the traverse

$\Sigma\Delta E_m$ = summation of the measured eastings within the traverse

$\Sigma\, l_i$ = summation of the distances within the traverse

l_{ij} = Distance from station i to station j.

The latitude (δN_e) and departure (δE_e) errors are added or subtracted from the measured latitudes (ΔN_m) and departures (ΔE_m). To obtain the adjusted latitudes (ΔN) and departures (ΔE), the algebraic signs (+/-) of the latitude (δN_e) and departure (δE_e) errors are reversed and the errors are added to the measured latitudes (ΔN_m) and departures (ΔE_m) (Table 8-1).

Adjusted Latitudes and Departures

$$\Delta N = \delta N_m \text{ +/- } \delta N_e$$

$$\Delta E = \delta E_m \text{ +/- } \delta E_e$$

e. Error detection. Errors in a balanced traverse are difficult to locate. Determining angular and linear errors in a balanced traverse requires inspection of the traverse plot and the line of closure (ρ). The perpendicular bisector of the line of closure (ρ) is utilized to find the angular error. If an angular error exists within the traverse, the perpendicular bisector of the line of closure will "point" (Figure 8-2) to the possible station that contains the error. The perpendicular bisector of the line of closure in Figure 8-2 represents a possible angular error in station A. The line of closure (ρ) may parallel a line within the traverse that may have a linear (distance) error (Figure 8-2). Figure 8-2 represents a possible distance error between stations A and B. These techniques in identifying angular and linear errors are effective when only one error exists within the traverse.

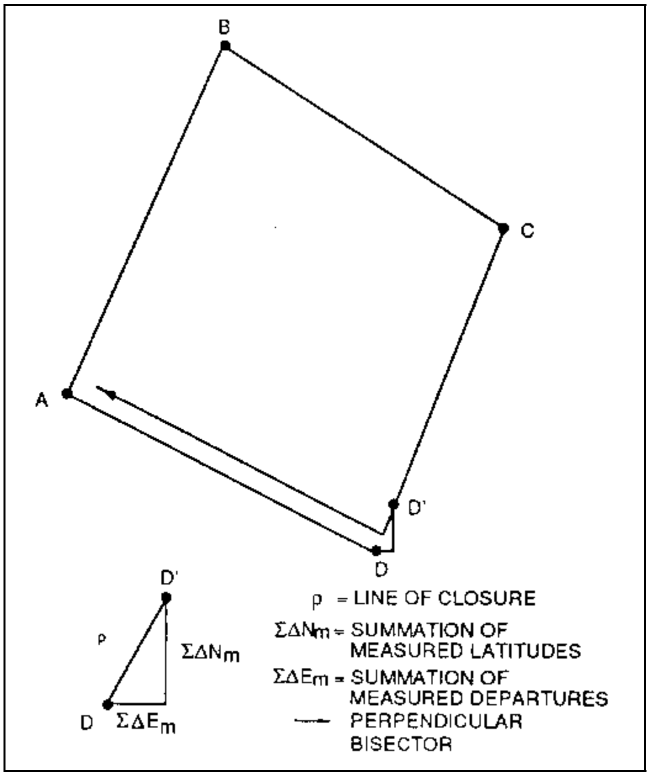


Figure 8-2. Error detection

8-4. Weighted Mean

The weighted mean (X_w) of a set of observations allows the surveyor to estimate the mean and variance of a parameter from a set of independent observations. The weight (w) of an observation is proportional to the inverse of the variance (σ^2) of the observation.

Observational Weight

$$w \approx 1/\sigma^2$$

Weighted Mean

$$X_w = (X_1w_1+X_2w_2+X_3w_3+X_nw_n)/(w_1+w_2+w_3+w_n\ )$$

X_w = weighted mean

X_1, X_2, X_n = independent observations

w_1, w_2, w_n = observational weights

If the position of a hydrographic vessel was determined by the two techniques Differential Global Positioning System (DGPS) and range azimuth techniques, the concept of the weighted mean can be utilized to determine the most probable position (Table 8-2). The precision of the weighted mean equals the inverse of the sum of the observational weights.

$$\sigma^2 = 1/\ (w_1 + w_2 + ... + w_n\ )$$

8-5. Least Squares Adjustment

The method of least squares is the procedure of adjusting a set of observations that constitute an over-determined model (redundancy $\succ$ 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.

$$v^twv = \phi$$

v = observation residual

w = weight of observation

ϕ = minimized criteria

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

Table 8-2
Weighted Mean

System	N (meters)	σ_N	w_N	E (meters)	σ_E	w_E
DGPS	2120373.545	1.5	0.444	3617852.015	.8	1.563
Range azimuth	2120380.243	1.8	0.309	3617860.323	2.4	0.174
Weighted mean	N 2120376.294		σ_N 1.2	E 3617852.847		σ_E 0.7

$\hat{l} = l + v$

$\hat{l}$ = adjusted observation

l = observation

v = residual

a. 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 five hundred and forty degrees (540°). If the correct model is not determined, the adjusted observations will be in error.

b. 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 traverse balancing techniques and weighted means do not allow for this variability. Since observations are affected by various errors, it is essential that the proper statistical information is applied.

8-6. Observations, Blunders, and Systematic and Random Errors

a. 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.

b. 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.

c. 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 DME the user should correct the distance for meteorological effects (temperature, pressure, relative humidity).

d. 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.

8-7. Variances, Standard Deviations, and Weights

The least squares principle incorporates the functional and stochastic models. It is essential that the correct a priori observational weights or variances are computed before the adjustment process.

a. Variance and standard deviation. The measure of variability of a set of observations is the sample (s^2) or population variance (σ^2). The greater the variance, the greater the variability of the observations. If a sample of observations has a variance of zero ($s^2 = 0$), the values of all observations are equal. Since the population mean (μ) is seldom known, the sample variance (s^2) is computed utilizing the sample mean (**x**).

Sample Variance

$$s^2 = [\Sigma (x_i - \mathbf{x})^2] / (n - 1)$$

Population Variance

$$\sigma^2 = [\Sigma (x_i - \mu)^2] / N$$

where

s^2 = sample variance

σ^2 = population variance

x_i = observations (where i = 0 through n)

x = sample mean

μ = population mean

n = number of observations

N = number of elements within the population

x equals: $x = (\Sigma x_i)/n$

μ equals: $\mu = (\Sigma x_i)/N$

The sample (s) and population (σ) standard deviation are the square root of the sample or population variances. Table 8-3 shows a list of four horizontal angles using a ten-second ($10''$) theodolite, sample mean (**x**), sample variance (s^2), standard deviation (s), and observational weight (w). If the user did not calculate the variance, the fact that the theodolite was a ten-second (10") instrument could be used as a standard deviation of the observations. However, it is advised that the surveyor calculate the variability (s^2) of a set of observations. Computing the sample variance provides a more accurate representation of the statistical nature of the observations.

b. Weights. The weight of an observation may be determined by empirical formulas, intuition, or observational analysis. The concept of weight is dependent on the a priori knowledge of the observational variance (σ^2 or s^2). The greater the observational weight, the greater the confidence in that observation. The least squares adjustment technique can accommodate absolute or relative weighting. Absolute weights are known if the observational variances have been measured or determined empirically. Relative weights are derived by intuition.

(1) Absolute weights. Absolute weights are computed empirically or through observational analysis (Table 8-3). Empirical weights are the result of experimentation or mathematical derivations. For example, DME or GPS manufacturers provide the user an equipment accuracy based on distance. These empirically derived values can be utilized to determine the variance or weight of an observation or set of observations.

Empirical Weights

$$\sigma = 5 \text{ mm} + 2 \text{ ppm} * \text{distance}$$

mm = millimeters

ppm = parts per million

If a distance of a thousand meters (1000 m) was measured between two locations, the observational weight would equal:

$$\text{ppm} = \text{millimeters (mm)/kilometers (km)}$$

$$1 \text{ km} = 1{,}000 \text{ m}$$

$$\sigma = 5 \text{ mm} + (2 \text{ mm/km}) * (1 \text{ km})$$

$$\sigma = 7 \text{ mm}$$

$$w = 1/\sigma^2$$

$$w = 0.02$$

(2) Relative weighting. Relative weighting is the result of intuition. In relative weighting, the user assigns weights based on past procedures, human factors, or physical phenomena. In level loop adjustments, relative weights (w) are considered inversely proportional to the leveled distance between stations. If the difference in elevation (de_{AB} = 0.512 m) was measured between stations A and B and the distance between the two stations was scaled from a map to be five hundred meters (500 m), the distance between the two stations could be used to compute the weight for the difference in elevation between stations A and B.

Table 8-3
Mean, Variance, Standard Deviation, and Weight

Observations	Deg	Min	Sec
1	142	20	20
2	142	20	30
3	142	20	10
4	142	20	10
Mean (**x**)	142	20	17.5
Variance(s^2)	000	01	31.6
Standard Deviation(s)	000	01	09.8
Observation Weight (w) $w = 1/s^2$			0.01

Relative Weights

$$w_{AB} \approx 1/d_{AB}$$

w_{AB} = observational weight

d_{AB} = distance between leveling stations

$$w_{AB} \approx 1/(500\ m)$$

$$w_{AB} \approx 0.002$$

8-8. Accuracy and Precision

The terms accuracy and precision are many times considered synonymous. However, they are unique, and the surveyor should use great care in how they are used to define a set of observations or coordinate values.

a. Accuracy. The accuracy of an observation is its degree of "closeness" to the true value (Figure 8-3). The RMS error statistic is often used to describe the accuracy of a set of observations. The RMS is centered about the true value and the standard deviation (σ) is centered about the mean value (**x**). The difference between the RMS and the standard deviation is the result of a bias between the measured and true value. This bias may be the result of systematic errors that were not removed prior to adjustment.

$$e = m - t$$

$$RMS = (\Sigma e^2/n)^{0.5}$$

where

Σe^2 = summation of the observational errors

m = measured value

t = true value

n = number of observations

RMS = root mean square error

b. Precision. The precision of an observation is its degree of closeness to the mean value (Figure 8-3). The variance or standard deviation is used to determine the precision of a set of observations.

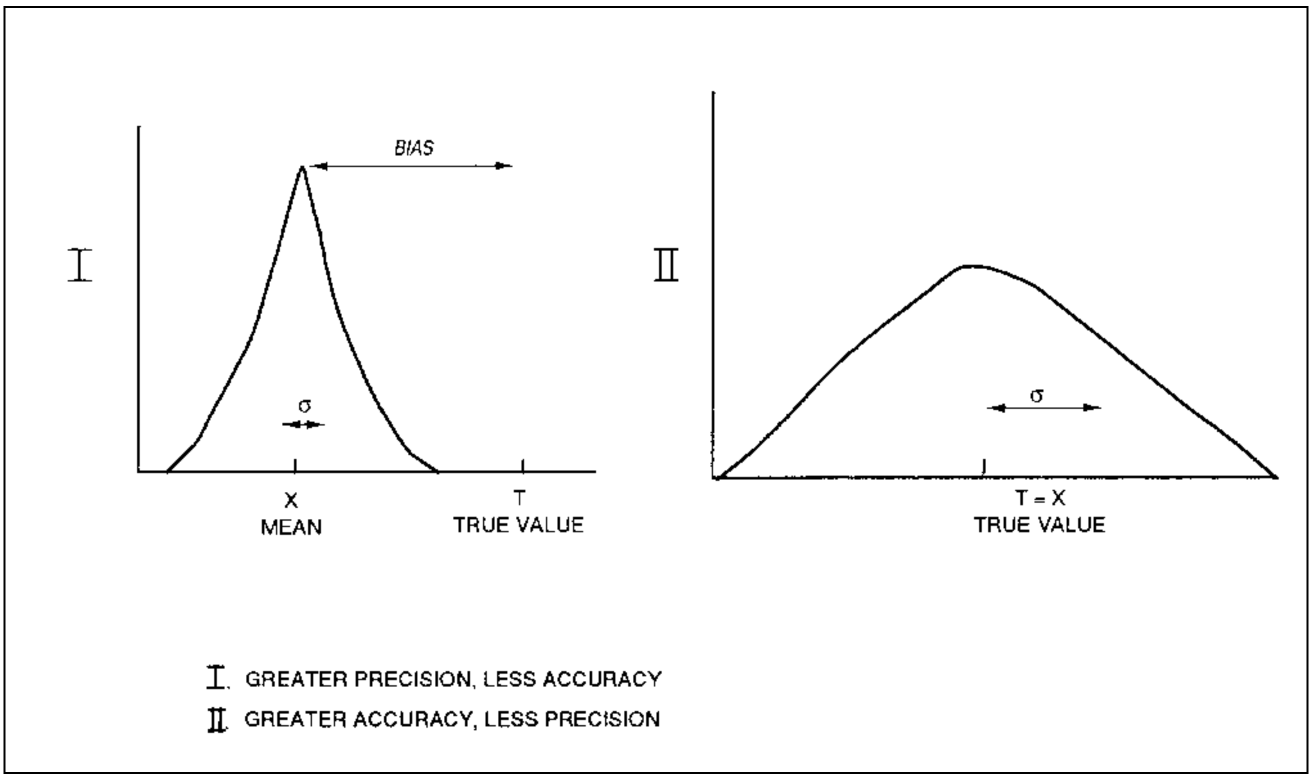


Figure 8-3. Precision and accuracy

8-9. Least Squares Adjustment Techniques

The user can employ various techniques in the adjustment of data using the least squares principle. The technique employed is dependent on the adjustment model, computational capability (computer resources), and the requirement of the survey. The reader should consult an introductory adjustment text to gain further understanding of the principles of adjustment theory. To fully understand the procedure of adjustment, a thorough understanding of matrix algebra and differential and integral calculus is required.

a. Adjustment model. The adjustment model consists of determining the number of observations to be adjusted (n), the minimum number of observations required to uniquely determine the functional model (n_o), and the redundancy (r). The model is determined by mathematical or physical relationships. For example, if the distances between three stations A, B, and C are to be determined (Figure 8-4a), the minimum number (n_o) of observations (distances) to fix the model are two. If the distance A to B (Figure 8-4b) was measured nine times and the distance B to C was not measured, the model could not be determined if the objective was to adjust the distance between A and C. Therefore, it is not only important to have redundant observations, but it is critical to have the correct number of observations to fix the model (n_o).

b. Observations only (Av = f). This method is seldom utilized because generalized software packages are difficult to develop. The method involves creating a condition or set of conditions that satisfies the functional model. Figure 8-5 shows a level loop involving three stations. Table 8-4 includes the differences in elevations and distances between stations A, B, and C. To perform a least squares adjustment for this level network, the adjustment model must be determined (n,n_o,r). The number of observations are the difference in elevations between the points (de_{ab},de_{bc},de_{ca}). If one station has a known elevation, two observations or difference in elevations are required to fix the adjustment model (n_o). If two stations have known elevations, one observation or difference in elevations is required to fix the adjustment model (n_o). In general, the minimum number of observations required to uniquely determine a level loop equals the number of stations in the loop minus the number of known stations. The redundancy for the model equals the number of observations minus the minimum number of observations to fix the model ($r = n - n_o$). The observations-only technique involves the use of condition

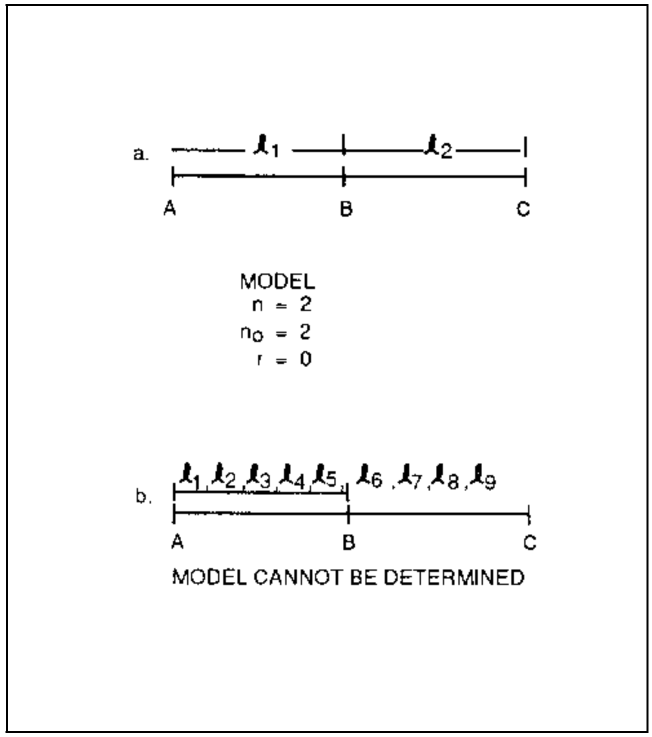


Figure 8-4. Adjustment model

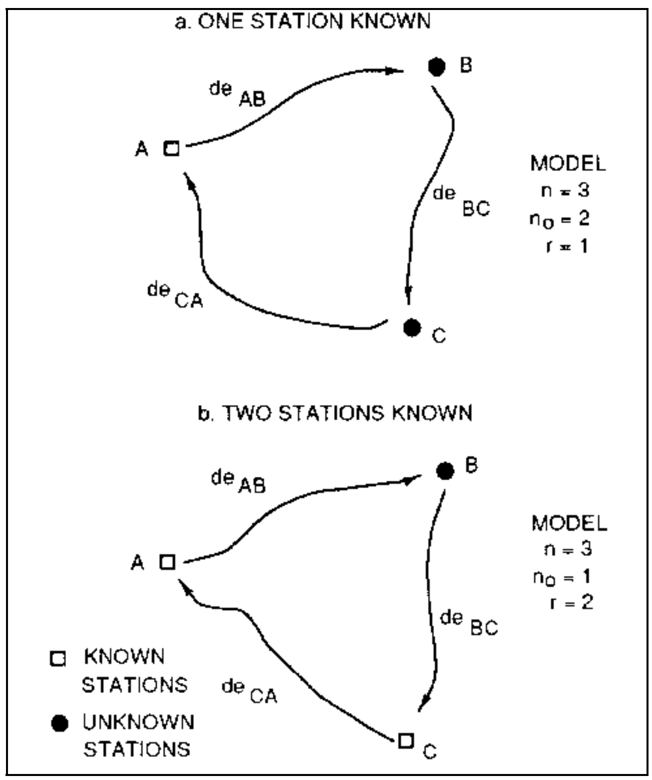


Figure 8-5. Level loop

Table 8-4
Level Loop

Known Elevation A = 232.150 m Adjusted Elevation B = ? Adjusted Elevation C = ?			
From	To	Difference in Elevation (DE) (meters)	Distance (meters)
A	B	-10.234	500
B	C	2.324	1200
C	A	7.821	1000

equations. In this example the elevation of one station (A) is considered known, the redundancy (r) equals one, and the condition equation equals the summation of the elevation differences from A, B, and C ($de_{ab} + de_{bc} + de_{ca} = 0$). In the observations-only technique, the number of condition equation equals the redundancy of the model.

Condition Equation

$$C = de_{ab} + de_{bc} + de_{ca} = 0$$

Step 1. Determine the number of observation (n), the minimum number of observations (n_o), and the redundancy ($r = n - n_o$) that satisfies the functional model.

Adjustment Model

$n = 3$

$n_o = 2$

$r = 1$

Compute the cofactor matrix (Q) which is the inverse of the weight matrix. In the level loop example the weight matrix is derived using the distance between leveling stations. The weight of each of observation is inversely proportional to the leveled distance. The dimensions of the cofactor and weight matrix are n x n (3 x 3).

where

n = number of observations

$$Q = W^{-1}$$

Q = cofactor matrix

W = weight matrix

$$W = \begin{bmatrix} \frac{1}{500} & 0.0 & 0.0 \\ 0.0 & \frac{1}{1200} & 0.0 \\ 0.0 & 0.0 & \frac{1}{1000} \end{bmatrix}$$

$$Q = \begin{pmatrix} 500 & 0 & 0 \\ 0 & 1200 & 0 \\ 0 & 0 & 1000 \end{pmatrix}$$

Step 2. Formulate the design matrix **A** and the misclosure vector **f**. The design matrix and misclosure vector have dimensions c x n (1 x 3) and c x 1 (1 x 1).

where

c = number of condition equations

n = number of observations

If the condition equations are linear, the elements of the design matrix (A) equal the coefficients of each observation in the condition equation. If the condition equations are nonlinear, they must be linearized using a Taylor series expansion. The condition equations for the level loop example are linear. The misclosure vector (f) equals the negative of the condition equations.

$$C = de_{ab} + de_{bc} + de_{ca} = 0$$

$$A = [\ 1\ 1\ 1]$$

$$f = [-de_{ab}\ - de_{bc}\ - de_{ca}]$$

$$f = [\ 10.234\ - 2.324 - 7.821]$$

$$f = [\ 0.089]$$

Step 3. The observational residuals are computed by the following matrix multiplications.

$k = (AQA^t)^{-1}f$

$k = [3.2963E^{-5}]$

$v = QA^t k$

$v = [\ 0.016\ 0.040\ 0.033]^t$

Step 4. The adjusted observations ($l^\wedge$) are computed by adding the post-adjustment residuals (v) to the measured observations.

$$de^\wedge = de + v$$

$$de^\wedge = \begin{pmatrix} -10.234 \\ 2.324 \\ 7.821 \end{pmatrix} + \begin{pmatrix} 0.016 \\ 0.040 \\ 0.033 \end{pmatrix}$$

$$de^\wedge = \begin{pmatrix} -10.218 \\ 2.364 \\ 7.854 \end{pmatrix}$$

The adjusted observations ($de^\wedge$) are utilized to determine if the condition was satisfied. Computational errors or an incorrect functional model can cause the adjusted observations not to satisfy the condition equations.

$C = de_{ab} + de_{bc} + de_{ca} = 0$

$C = -10.218 + 2.364 + 7.854 = 0$

Step 5. The estimated elevations of stations **B** and **C** are computed using the known elevation of station **A** and the adjusted differences in elevation ($de^\wedge$).

Elevation Estimates

$B = A + de_{ab}$

$B = 232.150$ m + (-10.218 m)

$B = 221.932$ m

$C = A + de_{ac}$

$C = 232.150$ m + (-7.854 m)

$C = 224.296$ m

c. Indirect observations ($v + B\Delta = f$). The indirect method includes observations and user-defined parameters. Observation equations are developed using parameters in terms of one observation. This method is commonly employed by commercial software packages. Figure 8-6 represents a survey where three ranges (distances) were measured from known survey control stations (A, B, and C) to a hydrographic vessel (D). The unknown position of the vessel was computed using the method of indirect observations. A minimum of two ranges (distances) ($n_o = 2$) are required to determine the unknown location of the vessel. The total number of observations is three (n = 3) and the redundancy of the system equals one (r = 1). The following steps outline the method of adjustment using indirect observations. Table 8-5 contains the observations, standard deviations, and coordinate information for the problem illustrated in Figure 8-6.

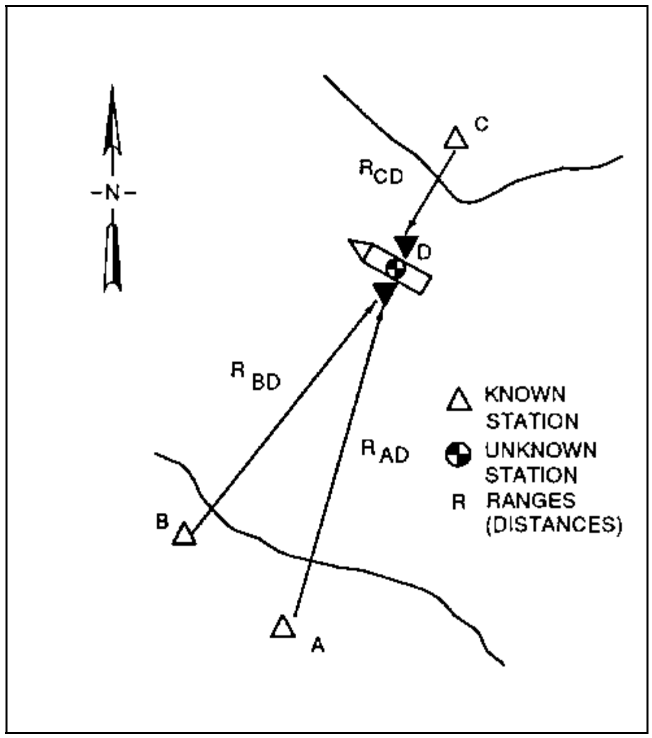


Figure 8-6. Intersection problem

Step 1. Determine the number of observations (n), the minimum number of observations (n_o), and the redundancy ($r = n - n_o$) that satisfies the functional model.

Adjustment Model

$n = 3$

$n_o = 2$

$r = 1$

Table 8-5
Intersection Example

Station	N (meters)	E (meters)	Range (R) to Station D (meters)	Range Standard Deviation (σ_R) (meters)
A	2111613.416	3616819.580	8562.825	.500
B	2117118.247	3610160.570	8238.020	.600
C	2120373.830	3617852.048	258.714	0.5

The weight matrix is (w) computed by using the stochastic properties of the measured ranges (σ_R). The weight is considered to be inversely proportional to the variance.

$$W = \begin{bmatrix} \frac{1}{(\sigma^2)_{ad}} & 0 & 0 \\ 0 & \frac{1}{(\sigma^2)_{ad}} & 0 \\ 0 & 0 & \frac{1}{(\sigma^2)_{ad}} \end{bmatrix} \quad W = \begin{pmatrix} 4.0 & 0 & 0 \\ 0 & 3.0 & 0 \\ 0 & 0 & 4.0 \end{pmatrix}$$

Step 2. In the method of indirect observations (v + BΔ = f) the number of condition or observation equations is equal to the total number of observations (n = 3). The observation equations (F_n) are developed such that they satisfy the functional model. The distance equation satisfies the functional model for the example in Figure 8-6.

Observation Equations

$$F_1 = R_{AD} - [\ (N_D - N_A)^2 + (E_D - E_A)^2\ ]^{0.5} = 0$$

$$F_2 = R_{BD} - [\ (N_D - N_B)^2 + (E_D - E_B)^2\ ]^{0.5} = 0$$

$$F_3 = R_{CD} - [\ (N_D - N_C)^2 + (E_D - E_C)^2\ ]^{0.5} = 0$$

Each observation equation (F_1,F_2,F_3) must be linearized with respect to the unknown parameters (N_D, E_D) using a Taylor series expansion. The design matrix (B) is comprised of the linearized observation equations. The design matrix has dimensions n x n_o (3 x 2). The nonlinearity of the problem requires that initial approximations are computed for the unknown parameters. The initial approximations ($N^o{}_D$, $E^o{}_D$) for the hydrographic vessel were computed from stations **A** and **B** using the technique of range-range intersection.

$$N^o{}_D = 2{,}120{,}115 \text{ meters}$$

$$E^o{}_D = 3{,}617{,}833 \text{ meters}$$

Design Matrix

$$B = \begin{bmatrix} \frac{\delta F_1}{\delta N_D} & \frac{\delta F_1}{\delta E_D} \\ \frac{\delta F_2}{\delta N_D} & \frac{\delta F_2}{\delta E_d} \\ \frac{\delta F_3}{\delta N_D} & \frac{\delta F_3}{\delta E_D} \end{bmatrix} \quad \frac{\delta F}{\delta N_D} = \frac{N_A - N_d}{\sqrt{(N_D - N_A)^2 + (E_D\ E_A)^2}} \quad \frac{\delta F}{\delta E_D} = \frac{E_A - E_D}{\sqrt{(N_D - N_A)^2 + (E_D - E_A)^2}}$$

The numerical elements of the design matrix (B) are computed by evaluating the partial derivatives ($\delta F_n/\delta N_D$, $\delta F_n/\delta E_D$) of the observation equations at the initial approximations ($N^o{}_D$, $E^o{}_D$) of the unknown parameters. The misclosure vector equals the negative of the observation equations. The misclosure vector (f) has dimensions n x 1 (3 x 1).

Misclosure Vector

$$f = \begin{matrix} F_1 = [\ (N_D - N_A)^2 + (E_D - E_A)^2\ ]^{0.5} - R_{AD} \\ F_2 = [\ (N_D - N_B)^2 + (E_D - E_B)^2\ ]^{0.5} - R_{BD} \\ F_3 = [\ (N_D - N_C)^2 + (E_D - E_C)^2\ ]^{0.5} - R_{CD} \end{matrix}$$

$$B{:}\ = \begin{pmatrix} -.9930 & -.1184 \\ -.3638 & -.9315 \\ .9973 & .0734 \end{pmatrix}$$

$$f{:}\ = \begin{pmatrix} -.1.053 \\ -1.109 \\ -0.184 \end{pmatrix}$$

Step 3. The estimates of the unknown parameters (Δ) and the residuals are computed by the following matrix manipulations. Since the problem is non-linear the algorithm must be repeated (iterated) until the solution converges. This example required two iterations to converge. The number of iterations is a function of data quality, initial approximations, and the functional model. The condition that the sum of the squares of the residuals

($v^t wv$) equals a minimum (ϕ) is utilized as a convergence criterion. The parameter vector (Δ) has dimensions n_o x 1 (2 x 1). In nonlinear problems the parameter vector must be added to the initial approximations to obtain the adjusted estimates.

Iteration 1

$$\Delta = (B^t WB)^{-1} * B^t Wf$$

$$\Delta := \begin{pmatrix} 0.323 \\ 1.107 \end{pmatrix}$$

$N^1_D = N^o_D + \Delta(1,1) = 2{,}120{,}115 \text{ m} + 0.323 \text{ m} = 2{,}120{,}115.323 \text{ m}$

$E^1_D = E^o_D + \Delta(2,1) = 3{,}617{,}833 \text{ m} + 1.107 \text{ m} = 3{,}617{,}834.107 \text{ m}$

$$v = f - B\Delta$$

$$v := \begin{pmatrix} -0.601 \\ 0.040 \\ -5.87 \end{pmatrix}$$

$$\phi_1 = v^t Wv = 2.83$$

Iteration 2

$$B := \begin{pmatrix} -.9930 & -.1185 \\ -.3638 & -.9315 \\ .9976 & .0694 \end{pmatrix}$$

$$f := \begin{pmatrix} -0.601 \\ 0.04 \\ -0.584 \end{pmatrix}$$

$$\Delta := \begin{pmatrix} 0.001 \\ 0.004 \end{pmatrix}$$

$$\phi_2 = 2.81$$

If $| \phi_1 - \phi_2|/\phi_1 \prec 0.01$ terminate adjustment

$(2.83 - 2.81)/2.83 = 0.007$

$0.007 \prec 0.01$

Final Adjusted Coordinates of Station D

$N_D = N^1 + \Delta(1,1) = 2{,}120{,}115.323 + 0.001 = 2{,}120{,}115.324 \text{ m}$

$E_D = E^1 + \Delta(2,1) = 3{,}617{,}834.107 + 0.004 + 3{,}617{,}834.111 \text{ m}$

d. General least squares ($Av + B\Delta = f$). This is the general case for the observations-only and indirect method. In some problems, equations that satisfy the functional model using the observations-only or indirect observations method may be difficult to develop. In the general case the condition equations can contain both parameters and multiple observations. The algorithm for this technique can be obtained by consulting an introductory adjustment textbook.

8-10. Error Analysis

After the adjustment procedure, the data are examined for observational blunders. Blunders and systematic errors should be identified and removed before the adjustment. However, erroneous observations are not always recognized before the initial adjustment procedure. To ensure that the final estimates are "free" of blunders, a blunder detection scheme should be implemented. The common blunder detection techniques are the global variance test, data snooping method, tau test, and robust estimation. Commercial software packages commonly employ the global and tau tests for the determination of blunders.

a. Statistical inference. Statistical inference involves the statement of a hypothesis. Statistical inference is most commonly utilized to identify observational blunders within the adjustment. The adjusted data are tested or compared to determine if the hypothesis is satisfied.

Hypothesis

H_o: Adjustment estimates are "free" of blunders

H_a Blunders

Four outcomes are possible from hypothesis testing:

(1) Select the null hypothesis (H_o), when the null hypothesis is true (correct decision).

(2) Select the alternative hypothesis (H_a), when the alternative hypothesis is true (correct decision).

(3) Select the alternative hypothesis (H_a), when in fact the null hypothesis (H_o) is true (type I error).

(4) Select the null hypothesis (H_o), when in fact the alternative hypothesis (H_a) is true (type II error).

The significance value (α) is dependent on the probability of committing a type I error. The probability of committing a type II error (1- β) is the result of accepting the null hypothesis when in fact the alternative hypothesis is true. The user must determine which error (type I or type II) will be the most costly. A significance level that is very large (α= 0.1) would decrease the confidence level (1-α). The smaller confidence level could result in the possible rejection of "good" observations. Therefore, if a system had limited redundancy, a smaller significance level may be warranted (α = 0.005).

b. Global variance test. Some software packages provide the user the opportunity to input a significance level or probability value. The a posteriori reference variance is one of the results of adjustment. If the a priori reference variance is known, the a posteriori variance is tested to determine if it is consistent with the a priori variance. The global test is applicable only when absolute weights were utilized in the adjustment. A two-tailed or upper tail test is constructed to test the variances.

A Posteriori Reference Variance

$$\sigma^2 = (v^t wv)/r$$

v^t = residual transposed

v = residual

r = redundancy

Two-Tailed Test

$$H_o : \sigma^2 = \sigma_o^2$$

$$H_a : \sigma^2 \neq \sigma_o^2$$

The two-tailed test fails if

$$X_r^2 < X^2_{\alpha/2,r} \text{ or } X_r^2 > X^2_{1-\alpha/2,r}.$$

Upper Tail Test

$$H_o : \sigma^2 = \sigma_o^2$$

$$H_a : \sigma^2 > \sigma_o^2$$

The upper tail test fails if $X_r^2 > X^2_{\alpha,r}$

$$X_r^2 = v^t wv/\sigma_o^2 = r\sigma^2/\sigma_o^2$$

$X^2_{\alpha,r}$, $X^2_{\alpha/2,r}$, $X^2_{1-\alpha/2,r}$ are critical values that are calculated from the chi square distribution based on a significance level alpha (α) and redundancy (r).

The failure of the global test suggests the possibility of a blunder; however, it does not identify the location of the blunder. Failure of the global test may also be the result of incorrect weighting or an incorrect adjustment model. The global test should be used in conjunction with the data snooping method or robust estimation. If an empirical method is used to determine the weights, the user must determine if the computed values are realistic. Otherwise, the global test may fail due to incorrect a priori weights. If the global test is rejected, the data require inspection for blunders or incorrect weighting. If the a posteriori reference variance passes the global test, it does not guarantee the absence of blunders within the adjustment. Therefore, the global test should only be used in conjunction with another analysis method.

c. Data snooping. The data snooping method requires the user to know the a priori reference variance. The method is very effective when only one blunder is in the network. The technique of data snooping sequentially tests each standardized residual within the adjustment and determines if it exceeds a defined rejection threshold. The standardized residual (v') is defined as the observational residual divided by the residual's standard deviation. The rejection threshold is computed based on a given significance level (α_o) using the Fisher distribution.

Standardized Residual

$$v' = v/\sigma_v$$

v' = standardized residual

$\sigma_v = (\sigma_o) * (q_{vi})^{0.5}$

σ_o = a priori reference standard deviation

q_{vi} = the ith diagonal element of the residual cofactor matrix (Q_{vv})

$Q_{vv} = Q - B(B^t WB)^{-1}B^t$

Data Snooping Rejection Criteria

$$|v'| > (F_{(1-\alpha o)},1,oo)^{0.5}$$

$$\text{If } |v'| > 3.29 \quad @ \ \alpha_o = 0.001$$

If the standardized residual exceeds the rejection threshold, it is considered an outlier. The suggested significance value for the data snooping technique is $\alpha_o = 0.001$. The rejection threshold corresponding to a significance value of 0.001 is 3.29. Therefore, the absolute value of all standardized residuals (v') that exceed 3.29 are identified as possible blunders. The data snooping technique is a univariate test that is very effective when only one blunder is in the network. Therefore, only the standardized residual with the greatest value that exceeds the rejection threshold is removed from the observations. After removal of the possible blunder, the adjustment is re-computed and the standardized residuals are tested for additional blunders. The procedure is continued until all blunders have been removed from the network.

d. Tau test. The tau test is used when the a priori reference variance is unknown. The tau test utilizes the tau distribution to compute the critical values. The tau ($\underline{t}$) distribution can be derived from the **t** (student) distribution using the following formula:

Tau Distribution

$$\underline{t} = [(r)^{0.5} * \mathbf{t}_{r-1}]/[(r-1 + \mathbf{t}^2_{r-1})^{0.5}]$$

r = redundancy

t = critical value from the **t** (student) distribution based on a significance level (α)

The rejection threshold is computed given a significance value (α) and the adjustment redundancy (r). The threshold is compared to the standardized residual. Since the a priori reference variance is not known, the standardized residual is computed using the a posteriori reference variance (σ^2). If the standardized residual exceeds a rejection threshold based on the tau distribution, the observation is flagged as a blunder. The tau rejection thresholds are interpolated from tables or generated from computer subroutines. After removal of the blunder, the adjustment is re-computed and the standardized residuals are tested for additional blunders. The procedure is continued until all blunders have been removed from the network.

Standardized Residual

$$v' = v/\sigma\ (q_{vi})^{0.5}$$

v' = standardized residual

σ = a posteriori standard deviation

q_{vi} = residual of the ith cofactor element

Tau Test Rejection Threshold

$$\text{If } v' \succ \underline{t}$$

Both the data snooping technique and tau test require the computation of the residual cofactor matrix (Q_{vv}). The determination of the residual cofactor matrix is computationally time consuming. To alleviate this time inefficiency, the diagonal components can be computed. This is only viable when the observational weight matrix is block diagonal. Another approach is the replacement of the standard deviation of the residual (σ_v) with the observational standard deviation (σ_l). The residual standard deviation is smaller than the observational standard deviation.

$$|v|/\sigma_l \prec |v|/\sigma_v$$

Therefore, an observation with a blunder may not be flagged as a possible erroneous measurement because the standardized residual will be smaller. If the observational standard deviation (σ_l) is substituted for the residual standard deviation (σ_v), it is recommended that the significance value (α) be increased. Increasing the significance value (α) will cause a decrease in the confidence level.

e. Robust estimation. The technique of robust estimation does not depend on the residual cofactor matrix (Q_{vv}) or the significance value (α). If the a priori reference variance is known, it is recommended that a global variance test be performed for the first adjustment iteration. The global test provides additional information on the presence of blunders or inaccuracies in the adjustment model or a priori weights. The robust estimation procedure changes the observational weights during each iteration for those observations that exceed a predefined threshold. The absolute value of the residuals divided by the observational standard deviations ($|v|/\sigma_l$) are sequentially analyzed to determine if they exceed a rejection threshold. If the residual exceeds the rejection threshold, the observational weight is reduced using a decreasing weight function. In essence, this technique removes observations with large residuals from the adjustment. The adjustment is continued until the solution converges. When the adjustment is completed all blunders should have been removed. The following decreasing weight functions are commonly utilized in the technique of robust estimation.

Weighting Function 1

All iterations:

$w^{i+1} = w^{i}\exp^{-|vi/\sigma i|}$ if $|v| \succ 3$

$w^{i+1} = w^{i}$ if $|v| \succ 3$

Weighting Function 2

Iteration 1

$w^{i+1} = w^{i}$

Iteration 2 & 3

$w^{i+1} = w^{i}(\exp[(|v_i/\sigma_i|)^{4.4}])^{0.05}$ if $|v| \succ 3$

$w^{i+1} = w^{i}$ if $|v| \succ 3$

following iterations:

$w^{i+1} = (\exp[-(|v_i/\sigma_i|^{3}])^{0.05}$

w^{i} = the weight of the ith observation

exp = exponential function

v_i = observational residual

σ_i = observational standard deviation

8-11. Interpretation and Analysis of Adjustment Results

The interpretation of adjustment results does not require knowledge of adjustment theory or advanced mathematics. The following section provides various "thumb rules" that can be utilized to determine the quality and reliability of the adjusted data. Although many software routines provide error analysis or blunder detection options the user must carefully interpret the results of these techniques. Data should not be rejected solely on the results provided by these packages.

a. Input parameters.

(1) Significance level (α). If the adjustment software provides the flexibility of inputting a significance level (α) the user should choose a value of 0.05. This value minimizes the probability of committing a type I and type II error.

(2) Initial coordinate estimates. Some programs require the user to input initial coordinate estimates. It is imperative that these values be realistic. If the initial estimates are erroneous, the adjustment may not converge to the correct solution. Techniques like traverse balancing and the weighted mean should be utilized to determine initial coordinates.

(3) Data input. All data must be entered in the correct linear and angular units. Never input one variable (coordinates) in feet and another variable (distances) in meters. Most software packages cannot accommodate mismatched units.

(4) Checking. Before the adjustment is computed, all field, office, and computer generated input should be checked by two individuals for blunders. Also, all known systematic errors should be removed from the data (i.e., meteorological data, collimation, and leveling corrections).

b. Output. The majority of the manufacturers' output the a posteriori reference variance, standardized residuals, and error ellipse information. Many software routines have blunder detection schemes that "flag" and remove observations that are possible blunders. These techniques of blunder detection are based on statistical tests like tau or data snooping. The drawback with these methods is they are many times unreliable. Therefore, the user must develop a "horse sense" in the determination and identification of blunders.

(1) Global test. The global test is utilized to determine blunders. However, if absolute weights were not used in the adjustment the results of the test are meaningless.

(2) Standardized residuals. The standardized residuals are an excellent indicator in determining blunders. After all blunders have been identified and removed using the manufacturers' software the output should be examined for additional blunders that were not located.

(3) Error ellipse.

f. Error ellipse. The error ellipse provides a representation of the precision of the adjusted parameters and observations. The error ellipse consists of semimajor (a) and semiminor axes (b). The semimajor (a) and semiminor (b) axes are precision estimates (σ) of the adjusted parameters. The error ellipse can be utilized to determine the absolute or relative precision of parameters.

8-12. Contract Monitoring

a. Recommendations. Contracts involving surveys that are small in areal extent and require low accuracy survey control (1:5,000) may not warrant a least squares adjustment. In general, Fourth-Order surveys (1:5,000) do not warrant adjustment. A compass rule adjustment or observational mean will suffice. All control surveys that are to be incorporated into the NGRS shall be performed and adjusted using the guidelines established by the NGS.

b. Required submittal documents. The contracting officer should require the contractor to supply the final adjustment for each project. The contractor should be required to supply a list containing any observations that were removed due to blunders. The contractor must provide the Corps with a detailed analysis explaining the methodology performed in the adjustment, assumptions, and possible error sources.

Chapter 9
Structural Deformation Monitoring Surveys

9-1. General

The Corps of Engineers has constructed hundreds of dams, locks, levees, and other flood control structures which require periodic surveys to monitor long-term movements and settlements or short-term deflections and deformations. In USACE, these types of surveys are generally referred to as "PICES Surveys" -- an acronym which derives from the directive Engineer Regulation ER 1110-2-100, Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures. This chapter provides background guidance on the general requirements for these surveys and describes some of the instrumentation and techniques available with which to perform deformation surveys. Subsequent chapters provide specific accuracy standards, specifications, frequencies, and procedural methods for performing various types of surveys.

a. Structural deformation. Dams, locks, levees, embankments, and other flood control structures are subject to external loads that cause deformation and permeation of the structure itself, as well as its foundations. Any indication of abnormal behavior may threaten the safety of the structure. Careful monitoring of the loads on a structure and its response to them can aid in determining abnormal behavior of that structure. In general, monitoring consists of both measurements and visual inspections, as outlined in ER 1110-2-100. To help to ensure the safe monitoring of a dam, it should be permanently equipped with proper instrumentation according to the goals of the observation, structure type and size, and site conditions. Guidance on the instrumentation required to measure internal loads on a structure can be found in USACE technical manuals dealing with concrete structures, earth and rock fill dams, and similar structures.

b. Concrete structures. It should be intuitive that deformations and periodic PICES observations will vary according to the type of structure. Differences in construction materials is one of the larger influences on how a structure deforms. For example, concrete dams deform differently than earthen or embankment dams. For concrete dams and other concrete flood control devices, deformation is mainly elastic and highly dependent on reservoir water pressure and temperature variations. Permanent deformation of the structure can sometimes occur as the subsoil adapts to new loads, concrete aging, or foundation rock fatigue. Such deformation is not considered unsafe if it does not go beyond a predetermined critical value. Therefore, PICES observations are typically configured to observing relatively long-term movement trends, including abnormal settlements, heaving, or lateral movements. Conventional geodetic survey methods from external points and of centimeter-level accuracy are sufficient to monitor these long-term trends. Highly accurate, short-term deflections or relative movements between monoliths due to varying temperature or hydraulic loading are more rarely required. These may include crack measurements or relative movements between monoliths over different hydraulic loadings. Relative movement deflections to the 0.01-inch accuracy level are common.

c. Earthen embankment structures. Earthen or embankment dams and levees obviously will deform altogether differently than concrete ones. With earthen dams, the deformation is largely characterized as more permanent. The self weight of the embankment and the hydrostatic pressure of the reservoir water largely force the fill material (and, in turn, the foundation if it too consists of soil) to settle, resulting in a vertical deflection of the structure. The reservoir water pressure also causes permanent horizontal deformation perpendicular to the embankment centerline. With earthen dams, elastic behavior is slight. PICES survey accuracy requirements are less rigid for earthen embankments, and traditional construction survey methods will usually provide sufficient accuracy. Typical PICES surveys include periodic measurement of embankment crest elevations and slopes to monitor settlements and slope stability. For embankment structures, survey accuracies of 0.1 foot are usually sufficient for monitoring long-term settlements and movements.

d. Long-term deformation monitoring. Depending on the type and condition of structure, PICES monitoring systems may need to be capable of measuring both long-term movement trends or short-term loading deformations. Long-term measurements are far more common and somewhat more complex given their external nature. Long-term monitoring of a structure's movement typically requires observations to monitoring points on the structure from external reference points. These external reference points are established on stable ground well removed from the structure or its construction influence. These external reference points are inter-connected and are termed the "Reference Network." The reference network must also be monitored at less-frequent intervals to ensure these reference points have not themselves moved. Traditional geodetic survey instruments and techniques may be employed to establish and monitor the reference network points, as described in Chapter 10. Procedures for observing periodic PICES observations from the reference points to the structure targets are covered in Chapter 12.

9-2. Deformation Monitoring Survey Techniques

a. The general procedures to monitor the deformation of a structure and its foundation involve measuring the spatial displacement of selected object points (i.e., target points) from reference points, themselves controlled in position. When the reference points are located in the structure themselves, only relative deformation can be determined. Micrometer joint measurements are relative observations. Absolute deformation or displacement can be determined if the reference points are located outside the actual structure, in the foundation or surrounding terrain, and beyond the area that may be affected by the dam or reservoir. Subsequent periodic observations are then made relative to these absolute reference points. Assessment of permanent deformations requires absolute data.

b. In general, for concrete dams it is ideal to place the reference points in a rock foundation at a depth unaffected by the reservoir. Once permanently monumented, these reference points can be easily accessed to perform deformation surveys with simple measurement devices. Fixed reference points located within the vicinity of the dam but outside the range of its impact are essential to determination of the deformation behavior of the structure. Thus, monitoring networks in the dam plane should be supplemented by and connected to triangulation networks and vertical control whenever possible.

c. The monitoring of dam or foundation deformation must be done in a manner such that the displacement is measured both horizontally and vertically (i.e., measurement along horizontal and vertical lines). Such measurements must include the foundation and extend as far as possible into it. Redundancy is essential in this form of deformation monitoring and is achieved through measuring at the points intersecting the orthogonal lines of the deformation network.

d. If a dam includes inspection galleries and shafts, deformation values along vertical lines can be obtained by using both hanging and/or inverted plumb lines and along horizontal lines by traverses; both are standard practice for deformation monitoring. Where there are no galleries or shafts (e.g., embankment dams, thin arch dams, or small gravity dams), the same result can be achieved by an orthogonal network of survey targets on the downstream face. These targets are sighted by angle measurements, typically combined with optical distance measurements, from reference points outside the dam.

e. A more routine, less costly, and more frequent monitoring process can be employed to monitor the short-term behavior of dams by simply confining observation to trends at selected points along the crest and sometimes vertical lines. Such procedures involve angle measurement or alignment (supplementing the measuring installation) along the crest to determine horizontal displacement, and leveling to determine vertical displacement. Even with this monitoring process, it is essential to extend leveling to some distance beyond the abutments. Alternative methods to that described include settlement gauges, hose leveling devices, or extensometers.

9-3. Accuracy Requirements for PICES Surveys

Table 9-1 provides general guidance on the accuracy requirements for performing PICES surveys. These represent either absolute or relative movement accuracies on structure target points that should be attained from survey observations made from external reference points. The accuracy by which the external reference network is established and periodically monitored for stability should exceed these accuracies. Many modern survey systems (e.g., DGPS, electronic total stations, bar-code levels, etc.) are easily capable of meeting or exceeding the accuracies shown below. However, PICES accuracy criteria must be defined relative to the particular structure's requirements, not the capabilities of a survey instrument or system.

a. For example, a good electronic total station system can measure movement in an embankment to the 0.005-foot level. Thus, a long-term creep of, say 3.085 feet, can be accurately measured. However, the only significant aspect of the 3.085-foot measurement is the fact that the embankment has sloughed "3.1 feet" -- the 0.001-foot resolution (precision) is not significant and should not be observed even if available with the equipment.

b. Relative crack or monolith joint micrometer measurements can be observed and recorded to 0.001-inch precisions. This precision is not necessarily representative of an absolute accuracy, given the overall error budget in the micrometer measurement system, measurement plugs, etc. Hydraulic load and temperature influences can radically change these short-term micrometer measurements at

Table 9-1
Typical Accuracy Requirements for PICES Surveys

Concrete Structures	
Dams, Outlet Works, Locks, Intake Structures:	
Long-term movement (Geodetic survey methods)	10 mm
Relative short-term deflections Crack/joint movements Monolith alignment (Precision micrometer alignments)	0.01 in. (0.2 mm)
Vertical stability/settlement (Precise geodetic leveling)	2 mm
Embankment Structures	
Earth-Rockfill Dams, Levees:	
Slope/crest stability (Total station/DGPS)	0.1 foot
Crest alignment (Total station/DGPS)	0.1 foot
Settlement measurements (Differential leveling)	0.05 foot
Control Structures	
Spillways, Stilling Basins, Approach/Outlet Channels, Reservoirs	
Scour/erosion/silting (Hydrographic surveys)	0.2 to 0.5 foot

the 0.01- to 0.02-inch level, or more. Attempts to observe and record micrometer measurements to a 0.001-inch precision with a ±0.01-inch temperature fluctuation is wasted effort.

9-4. PICES Classification for Determining Monitoring Frequency

The following presents general guidance in determining the frequency which periodic monitoring surveys must be performed, giving variable structure type, risk, and age.

Class I: HIGH RISK STRUCTURES IN DISTRESS. The high risk of Class I structures warrants continuous monitoring of the structure.

Type A: POTENTIAL FAILURE IMMINENT. Gather data as can/if can/if prudent. Data are very valuable for later analysis of why the structure failed. Use any method available to gather information and data without risk of life or interference in processes ongoing to save the structure and/or alert the population at risk.

Type B: POTENTIAL FAILURE SUSPECTED. Monitor structure continuously. After potential solution to save structure is applied, continue with continuous monitoring until it is determined that structure is stabilized.

Type C: DAMS OR RESERVOIRS UNDERGOING INITIAL IMPOUNDMENT. Gather ground truth data before impoundment procedures start. Monitor continuously until failure is suspected or until impoundment procedures have halted. Continue monitoring continuously until it is determined that structure has stabilized and will maintain as planned under load.

Class II: MEDIUM RISK STRUCTURES NOT IN DISTRESS.
Such structures are of a category of risk such that monitoring every other year is prudent. Structures of this category are stable, but whose failure would:

- affect a population area;
- result in a high dollar loss of downstream property;
- cause a devastating interruption of the services provided by the structure.

Type A: LARGE STRUCTURES.
i. Reservoirs.
ii. Dams with power plants.

Type B: SMALLER STRUCTURES.
i. Reservoirs and dams.
ii. Dams without power plants.
iii. Locks.

Class III: LOWER RISK STRUCTURES NOT IN DISTRESS.
Such structures are of a category of risk such that monitoring every other year is prudent. Structures of this category are stable, but whose failure would:

- not affect a population area;
- not result in a high dollar loss of downstream property;
- not cause a devastating interruption of the services provided by the structure.

Type A: LARGE STRUCTURES.
i. Reservoirs.
ii. Dams with power plants.

Type B: SMALLER STRUCTURES.
i. Reservoirs and dams.
ii. Dams without power plants.
iii. Locks.

Table 9-2 summarizes the PICES structure classification for determining monitoring frequency.

9-5. Dam Safety

a. Minimizing the risk of dam failure is a primary function of USACE employees involved with PICES work. Existing USACE dams and reservoirs must be periodically inspected so that their structural soundness can be evaluated and reevaluated.

b. In the course of time, dam structures may take on anisotropic characteristics. Internal pressures and paths of seepage may develop. Usually the changes are slow and not readily discerned by visual examination. Continuous monitoring of a dam's performance will usually ensure detection of any flaws which may lead to failure. This must be done by personnel who know the signs of distress. The best knowledge of the forces which cause failure are gained by studying the postmortems of the failed structures themselves.

c. Analysis of the performances for the various types of dams will show their relative suitability for conditions which may be encountered at a given site. Each type can be related generally to a certain mode of failure. A gravity dam may collapse only in the section which is overstressed. A buttress dam may fall in domino fashion through the successive collapse of its buttresses. The rupture of an arch may be sudden and complete. Failure of an embankment may be relatively slow, with erosion progressing laterally and downward and then accelerating as the floodwaters rip through the breach.

d. The records of dams indicate that earthfills have been involved in the largest number of failures, followed by gravity dams, rockfills, and multiple and single arches. Considering the percentages, however, the arch dams show a higher failure rate. Studies show that there are two predominant causes of damage which are:

- Construction defects.

- Interstitial water that is inadequately controlled.

9-6. Foundation Problems in Dams

a. Foundation problems may lead to breaching of the dam. Differential settlement, sliding, high piezometric pressures, and uncontrolled seepage are common evidences of foundation distress. Cracks in the dam, even minor ones, can indicate a foundation problem. Potential erosion of the foundation must be considered. Clay or silt in weathered joints can preclude grouting and eventually swell the crack enlarging it and causing further stress. Foundation seepage can cause internal erosion or solution. Erosion can leave collapsible voids. Actual deterioration may be evidenced by increased seepage, by sediment in seepage water, or an increase in soluble materials disclosed by chemical analyses. Materials vulnerable to such erosion include such materials as dispersive clays, water reactive shales, gypsum, and limestone.

b. Pumping from underground can cause foundation settlement as the supporting water pressure is removed or the gradient changed. Loading and wetting will also cause the pressure gradient to change, and may also cause settlement or shifting. The consequent cracking of the

Table 9-2
PICES Structure Classification

Structures in Distress		Structures Not In Distress			
Class I: High Risk		Class II: Medium Risk		Class III: Lower Risk	
Continuous Monitoring		Monitor Yearly or Every Other Year		Monitor Every Other Year	
Type A	Potential Failure Imminent	Type A	Large structures	Type A	Large structures
Type B	Potential Failure Suspected				
Type C	Dams or Reservoirs Undergoing Initial Impoundment	Type B	Smaller structures	Type B	Smaller structures

dam can create a dangerous condition, especially in earthfills of low cohesive strength.

c. Foundations with low shear strength or with seams of weak materials such as clay or bentonite may be vulnerable to sliding. Shear zones can also cause problems at dam sites. The bedding plane zones in sedimentary rocks and foliation zones in metamorphics are two common problems.

9-7. Seepage

a. Water movement through a dam or through its foundation is one of the important indicators of the condition of the structure and may be a serious source of trouble. Seeping water can chemically attack the components of the dam foundation. Constant attention must be focused on any changes such as in the rate of seepage, settlement, or in the character of the escaping water. Adequate measurements must be taken of the piezometric surface within the foundation and the embankment, as well as any horizontal or vertical distortion of the abutments and the fill. Generally, differential settlements caused by the dissolving of solid material develop slowly enough to provide advance warning of the need for any remedies.

b. Any leakage at an earth embankment may be potentially dangerous, since rapid erosion may quickly enlarge an initially minor defect.

9-8. Erosion

Embankments may be susceptible to erosion unless protected from wave action on the upstream face and surface runoff on the downstream face. Riprap armor stone on the upstream slope of an earthfill structure can protect against wave erosion, but can become dislodged due to wave action. This deficiency can usually be detected and corrected before serious damage occurs.

9-9. Embankment Movement

a. In an older embankment dam, the condition of materials may vary considerably. There may be small or extensive areas of low strength. Location of these weaknesses must be a key objective of the evaluation of such dams. Soluble materials are sometimes used in construction, and instability in the embankment will develop as these materials are dissolved over time.

b. Adverse conditions which deserve attention include: poorly sealed foundations, cracking in the core zone, cracking at zonal interfaces, soluble foundation rock, deteriorating impervious structural membranes, inadequate foundation cutoffs, desiccation of clay fill, steep slopes vulnerable to sliding, blocky foundation rock susceptible to differential settlement, ineffective contact at adjoining structures and at abutments, pervious embankment strata, vulnerability to conditions during an earthquake.

c. An embankment may be most vulnerable at its interface with rock abutments. Several dam failures have occurred during initial impoundment. Settlement in rockfill dams can be significantly reduced if the rockfill is mechanically compacted. In some ways, a compacted earth core is superior to a concrete slab as the impervious element of a rockfill dam. If the core has sufficient plasticity, it can be flexible enough to sustain pressures without significant damage.

9-10. Liquefaction

Liquefaction can occur during earthquakes. Hydraulic fill dams are particularly susceptible to this type of damage. Liquefaction is a potential problem for any embankment which has continuous layers of soil with uniform gradation and of fine grain size. The Fort Peck Dam experienced a massive slide on the upstream side in 1938, which brought the hydraulic fill dam under suspicion. The investigation at the time focused blame on an incompetent foundation, but few hydraulic fills were built after the 1930's. Heavy compaction equipment became available in the 1940's, and the rolled embankment dam became the competitive alternative.

9-11. Concrete Deterioration

Chemical and physical factors can age concrete. Visible clues to the deterioration include: expansion, cracking of random pattern, gelatinous discharge, and chalky surfaces.

9-12. PICES Measurements on Other USACE Navigation and Flood Control Structures

a. Concrete navigation lock monoliths and miter lock gates. The Corps operates approximately 270 navigation lock chambers constructed of plain or reinforced concrete. PICES surveys may be required over many of these structures to monitor potential problem areas. The frequency of these periodic surveys will be highly variable. Likewise, not all structural components of a lock complex (e.g., wall/monoliths, wing walls, gates, dam) may need to be monitored. These measurements are related to a list of 10 functional distresses. Observations for distresses in miter lock gates may include one or more of the following:

Top anchorage movement
Elevation change
Miter offset
Bearing gaps
Downstream movement
Cracks
Leaks/boils
Dents
Abnormal noise or vibration during gate operation
Corrosion

b. Sheet pile structures. PICES observations for distresses in sheet pile structures may include one or more of the following:

Misalignment
Corrosion
Settlement
Cavity formation
Interlock separation
Holes
Dents
Cracks

c. Rubble breakwaters and jetties. Any number of measurements may be needed to monitor the condition of breakwater and jetties. These may involve either conventional surveying or hydrographic methods. Typical observations include measurements for the seaside and leeside slopes and crest in each of the following categories:

Seaside/Leeside Slope:	Crest/Cap:
Armor loss	Breaching
Armor quality defects	Armor loss
Lack of armor contact/interlock	
Core exposure/loss	Core exposure/loss
Slope defects	

9-13. Deformations in Large Structures

Deformation of large structures (e.g., dams) is mainly caused in part by the reservoir level. Other factors also play a role in the deformation of the structure, including temperature, self-weight of the material from which the dam is constructed, and earth pressure. A monitoring system should therefore include regular measurements of the reservoir level and temperature and pressure data.

a. Reservoir level measurement. Reservoir levels today should be measured with pressure balances. Double checking the measurements must be done and can be facilitated by installing a manometer on either an existing or new pipe connected to the reservoir. The measurement range should extend at least as far as the dam crest, thereby allowing observation and judgment of the flood risk and assessment of peak inflows.

b. Temperature measurement. Temperature measurement is required to determine the impact of temperature variations on the structure itself, as well as whether precipitation consists of rain or snow and if applicable, whether the snow melt period has begun. Temperature measurement should be done at least daily. Double checking is not necessary, as other methods can be used if failure takes place. The thermometers should be placed at various locations within the dam, either embedded in the structure itself or within drillholes. Redundancy should be provided for by using a greater number of thermometers than otherwise would be required.

c. Precipitation measurement. Precipitation measurement should be done by using a precipitation gauge. Daily readings are recommended. The gauging station does not need to be located at the dam site, but should not be too far away so as to not be representative of the precipitation level at the structure itself. Redundancy is not necessary for precipitation measurement as gauges located further away can often be used when the closer one fails. Every large structure has some form of seepage through the structure itself or its foundation, even if it has a grout curtain. In concrete dams, seepage typically is small and limited to permeable areas of the concrete, joints, and contact between rock and concrete. Any abnormal seepage is an immediate warning that something may be wrong with the structure or foundation. Seepage flows cause uplift pressure - pressure which must be monitored in view of its critical impact on the overall stability of the structure. In embankment (i.e., earthen) dams, seepage flow through the structure itself is similar to that observed in its foundation as the material from which both are made are pervious. Seepage flows not only cause uplift pressure in these structures, but also pore-water pressure. The pattern of seepage and water pressures on the structure (especially on the foundation and impervious core) has a significant impact on the behavior of the dam. Therefore, seepage is critical and must be carefully monitored as any abnormal rate may indicate a condition which is a serious threat to the overall stability and safety of the structure.

d. Seepage rate. The total seepage rate is the seepage at the face of the structure taken as a whole. As a whole, such rates can be assessed as to whether they are "normal." Seepage rate can be measured volumetrically by using a calibrated container and a stopwatch or a gauging weir or flume. Such methods are straightforward, simple, and reliable; therefore redundancy is not necessary. Partial seepage rates are taken in isolated zones of the structure found to be representative for the area examined. Such rates should be monitored periodically. In the course of monitoring seepage, if an abnormality (i.e., change in normal seepage rate) is detected, the critical zone and cause of the seepage is easier to identify.

e. Chemical property analysis. If the structure is constructed of soluble or easily erodible material, the seepage should be monitored for turbidity and chemical content also. Doing so will permit the assessment of the overall stability of the embankment and foundation materials.

f. Pore-water pressure measurement. Structures usually are designed with specific pore-water pressure values that should not be exceeded. Pressure cells typically are designed or built into the structure themselves to measure pore-water pressure. The linking together of several cells forms a profile for the structure. The greater the number of measurement profiles and number of cells per profile, the more useful the data obtained will be. Even though pressure cells can be installed in structures themselves, rehabilitation of existing ones is not always practicable. Where pressure cells cannot be used to monitor pore-water pressure, the phreatic line in selected points will be monitored. Standpipe piezometers mounted in the embankment at several cross sections should be used to monitor the phreatic line.

g. Uplift pressure. Seepage underneath a structure causes uplift pressure which can severely alter the stabilizing effect of the structure's self weight. Uplift pressure can be reasonably controlled by a grout curtain and drainage holes, but uplift pressure and the physical effectiveness of these control measures should be carefully monitored. Piezometers connected to a manometer are a reliable means to measure the uplift pressure in cross-sections and several points on the upstream and downstream face of the structure.

h. Discharge measurement. If the foundation is being drained, drainage discharge should be monitored by either volumetric gauging or gauging weir. Such methods are sufficiently reliable not to require redundancy. Any change in flow rate may be indicative of clogging in the drainage system. If possible, the discharge of any spring, rivers, streams, flood control structure, etc. downstream of the structure should be monitored to gauge the effect of any discharge on them. Any variations in discharge may be indicative of a seepage problem.

9-14. Current Deformation Methods, Equipment, and Analysis Techniques

This section describes the latest technology used for performing both relative and absolute deformation surveys. Many of the procedures and methods contained herein require extensive geodetic survey background, and are applicable only to large, high head dams, or on structures with unique problems. They should not be employed on smaller structures or earthen embankments.

a. Deformation monitoring, analysis, and prediction are of a major and ever-growing concern in practically all fields of engineering and geoscience. Safety, economical design of man-made structures, efficient functioning and fitting of structural elements, environmental protection, and development of mitigative measures in the case of natural disasters (land slides, earthquakes, liquefaction of earth dams, etc.) require a good understanding of causes (loads) and the mechanism of deformation which can be achieved only through the proper monitoring and analysis of deformable bodies.

b. The development of new methods and techniques for the monitoring and analysis of deformations and the development of methods for the optimal modeling and prediction of deformations have been the subject of intensive studies by many professional groups at national and international levels. Within the most active international organizations which are involved in deformation studies one should list:

- International Federation of Surveyors (FIG) with its Study Group 6C which has significantly contributed to the recent development of new methods for the design and geometrical analysis of integrated deformation surveys and new concepts for global integrated analyses and modeling of deformations;

- International Commission on Large Dams (ICOLD) with its Committee on Monitoring of Dams and their Foundations;

- International Association of Geodesy (IAG) with the very active Commission on Recent Crustal Movements which frequently organizes international and regional symposia concerning geodynamics, tectonic plate movements, and modeling of regional earth crust deformation;

- International Society for Mine Surveying (ISM) with their very active Commission 4 on Ground Subsidence and Surface Protection in mining areas;

- International Society for Rock Mechanics (ISRM) with their overall interest in rock stability and ground control; and

- International Association of Hydrological Sciences (IAHS) which organizes international symposia (e.g., Venice 1984, Houston 1991) on ground subsidence due to the withdrawal of underground liquids (water, oil, etc.).

c. The FIG Study Group 6C has been one of the most, if not the most, active international groups dealing with practically all aspects of deformation monitoring and analysis. Since 1975, the FIG Study Group 6C has organized six international symposia with the last symposium held in Hanover in 1992. Although the activity of FIG in the development of new monitoring techniques is biased, of course, toward geodetic surveying techniques, its activity in the design and analysis of deformation surveys is more objective than that of any other professional group. In 1978, an ad hoc Committee on Deformation Analysis was formed to deal with and clarify various approaches and schools of thinking regarding the geometrical analysis of deformation surveys including the identification of unstable reference points. The work of the Committee has resulted in the development of new concepts for integrated monitoring systems (integration of geodetic and geotechnical measurements) and for generalized global analyses of integrated deformation surveys.

d. Most of the activities and studies of other associations and organizations focus, of course, on direct applications to their particular deformation problems. Although the accuracy and sensitivity criteria for the determination of deformation may considerably differ between various applications, the basic principles of the design of monitoring schemes and their geometrical analysis remain the same. For example, a study on the stability of magnets in a nuclear accelerator may require determination of relative displacements with an accuracy of 0.05 mm while a settlement study of a rock-fill dam may require only 10 mm or larger. In both cases, although the monitoring techniques and instrumentation may differ, one may show that the same basic methodology in the designing and analysis of the deformation measurements may be utilized. This fact has not yet been fully understood by most of the above-listed international research groups. There is a general lack of communication and work coordination. The studies of various professional groups, not only at the international but also at the national level of individual countries, overlap resulting in the duplication of efforts in discovering methods and techniques which have already been well known to other study groups. For example, in the United States there is very little communication between the U.S. Bureau of Reclamation, U.S. Army Corps of Engineers, U.S. Commission on Large Dams, and the American Congress on Surveying and Mapping. All four organizations are involved separately in studies on deformations of engineering structures and in the development of guidelines for deformation monitoring. Even within the same organization or institution, one may find examples of two different professional groups, for instance, geotechnical and structural engineers, who may work on the same deformable object but do not exchange information on their methods and the results of their analyses.

e. According to the most recent information obtained from the U.S. Committee on Large Dams, USACE is in charge of 475 large dams out of a total of 5,469 large dams maintained currently in the United States. An additional 49 large dams are currently under construction.

9-15. Review of Monitoring Techniques and Instrumentation

a. The monitoring schemes include:

- Monitoring of deformations, i.e., determination of the geometrical change in shape and dimensions and rigid body translations and rotations (absolute and/or relative) of the monitored object; and

- Monitoring of acting forces (loads) and internal stresses which can either be measured directly or derived from measurements of temperature, pore water pressure, water level, seepage, etc.

b. In addition, laboratory and/or in situ determinations of physical properties of the deformable material (e.g., moduli of elasticity, tensile strength, creep parameters, porosity, etc.) are necessary for a proper design and evaluation of the behavior of the monitored structure. In seismically active areas, the monitoring schemes must include special instrumentation for measuring vibrations.

c. This section reviews only the techniques used in monitoring the deformations although all the other above-mentioned components of the monitoring schemes play an equally important role in the analysis and interpretation of deformations as discussed below.

d. The measuring techniques and instrumentation for geometrical monitoring of deformations have traditionally been categorized into two groups according to the two main groups of professionals who use the techniques:

(1) Geodetic surveys which include conventional terrestrial, photogrammetric, satellite positioning, and some special techniques (interferometry, hydrostatic leveling, alignment, and other), and,

(2) Geotechnical/structural measurements of local deformations using tiltmeters, strainmeters, extensometers, joint meters, plumb lines, etc.

e. Each type of measurement has its own advantages and drawbacks. Geodetic surveys, through a network of points interconnected by angle and/or distance measurements, usually supply a sufficient redundancy of observations for the statistical evaluation of their quality and for a detection of errors. They give global information on the behavior of the deformable object while the geotechnical measurements give very localized and, very frequently, locally disturbed information without any check unless compared with some other independent measurements. On the other hand, geotechnical instruments are easier to adapt for automatic and continuous monitoring than conventional geodetic instruments. Conventional terrestrial surveys are labor intensive and require skilful observers, while geotechnical instruments, once installed, require only infrequent checks on their performance. Geodetic surveys have traditionally been used mainly for determining the absolute displacements of selected points on the surface of the object with respect to some reference points that are assumed to be stable. Geotechnical measurements have traditionally been used mainly for relative deformation measurements within the deformable object and its surroundings. However, with the technological progress of the last few years, the differences between the two techniques and their main applications are not as obvious as 20 years ago. For example, inverted plumb lines and borehole extensometers, if anchored deeply enough in bedrock below the deformation zone, may serve the same way as, or even better than, geodetic surveys for determining the absolute displacements of the object points. Geodetic surveys with optical and electromagnetic instruments (including satellite techniques) are always contaminated by atmospheric (tropospheric and ionospheric) refraction which limits their positioning accuracy to about ±1 ppm to ±2 ppm (at the standard deviation level) of the distance. So, for instance, with the average distance between the object and reference points of 500 m, the absolute displacements of the object points cannot be determined with an accuracy better than about ±2 mm at the 95 percent probability level. In some cases this accuracy is not adequate. On the other hand, precision electro-optical geodetic instruments for electronic distance measurements (EDM), with their accuracies of ±0.3 mm over short distances, may serve as extensometers in relative deformation surveys. Similarly, geodetic leveling, with an achievable accuracy of better than ±0.1 mm over distances of 20 m may provide better accuracy for the tilt determination (equivalent to ±1 second of arc) than any local measurements with electronic tiltmeters. New developments in three-dimensional coordinating systems with electronic theodolites may provide relative positioning in almost real time to an accuracy of ±0.05 mm over distances of several meters. The same applies to new developments in photogrammetric measurements with the solid state cameras (CCD sensors). The satellite GPS, which, if properly handled, offers a few millimeters accuracy in differential positioning over several kilometers, is replacing conventional terrestrial surveys in many deformation studies and, particularly, in establishing the reference networks.

f. A full review of all the instruments and techniques available for deformation monitoring would far exceed the scope of this manual. In view of the available references listed in Appendix A, the material below is limited only to some comments on the achievable accuracy of the basic types of instruments which include electronic instrumentation for distance and angle measurements, surveying robots, measurements of tilt and inclination, changes in distances and strain, photogrammetric methods, and the satellite GPS. It is assumed that the users of this manual possess the basic understanding of geodetic and geotechnical measuring techniques and do not require explanations of what is a theodolite, or a hydrostatic level, or a tiltmeter. Additional information can be found in the references in Appendix A.

9-16. Electronic Distance and Angle Measurements

a. Electronic theodolites. Over the last two decades, the technological progress in angle measurements has been mainly in the automation of the readout systems of the horizontal and vertical circles of the theodolites. The optical readout systems have been replaced by various, mainly photo-electronic, scanning systems of coded circles with an automatic digital display and transfer of the readout to electronic data collectors or computers. Either decimal units (gons) or traditional sexagesimal units of degrees, minutes, and seconds of arc may be selected for the readout. The sexagesimal system of angular units, which is still commonly accepted in North America, is used throughout this section. The relationship between the two systems is that 360˚ = 400 gons.

(1) As far as accuracy is concerned, electronic theodolites have not brought any drastic improvements in comparison with precision optical theodolites. Some of the precision electronic theodolites, such as the Kern E2 (discontinued production), Leica (Wild) T2002 and T3000, and a few others, are equipped with microprocessor controlled biaxial sensors (electronic tiltmeters) which can sense the inclination (misleveling) of the theodolite to an accuracy better than 0.5" and automatically correct not only vertical but also horizontal direction readouts. In optical theodolites in which the inclination is controlled only by a spirit level, errors of several seconds of arc in horizontal directions could be produced when observing along steeply inclined lines of sight. Therefore, when selecting an electronic theodolite for precision surveys, one should always choose one with the biaxial leveling compensator.

(2) Human errors of pointing the telescope to the target, centering errors, and environmental influences are the main factors limiting the achievable accuracy. The environmental influence of atmospheric refraction is a particular danger to any optical measurements. The gradient of air temperature, dT/dx, in the direction perpendicular to the line of sight is the main parameter of refraction. Assuming that a uniform temperature gradient persists over the whole length S of the line of sight, the refraction error e_{ref} of the observed direction may be approximately expressed in seconds of arc by:

$$e_{ref} = 8''(SP/T^2)dT/dx \qquad (9\text{-}1)$$

where P is the barometric pressure in millibars and T is the temperature in Kelvin (T = 273.15 + t˚C). If a gradient of only 0.1˚C/m persists over a distance of 500 m at P = 1000 mb and t = 27˚C, it will cause a directional error of 4.4", which is equivalent to a 12-mm positional error of the target. One should always avoid measurements close to any surface that may have a different temperature than the surrounding air (walls of structures or soil exposed to the sun's radiation, walls of deep tunnels, etc.). If any suspicion of a refraction influence arises, the surveys should be repeated in different environmental conditions in order to randomize its effect.

(3) Generally, with well designed targets and proper methodology, an accuracy (standard deviation) better than 1" in angle measurements can be achieved with precision electronic theodolites if three to five sets of observations are taken in two positions (direct and reverse) of the telescope. The requirement of two positions must always be obeyed in order to eliminate errors caused by mechanical misalignment of the theodolite's axial system. This applies to both the old and most of the up-to-date theodolites even if the manufacturer claims that the errors are taken care of automatically.

b. Three-dimensional coordinating systems. Two or more electronic theodolites linked to a microcomputer create a three-dimensional (3-D) coordinating (positioning) system with on-line calculations of the coordinates. The systems are used for the highest precision positioning and deformation monitoring surveys over small areas. Leica (Wild) TMS and UPM400 (Geotronics, Sweden) are examples of such systems. If standard deviations of simultaneously measured horizontal and vertical angles do not exceed 1", then positions (x, y, z) of targets at distances up to 10 m away may be determined with the standard deviations smaller than 0.05 mm. Usually short invar rods of known length are included in the measuring scheme to provide scale for the calculation of coordinates.

c. Electronic distance measurements (EDM). Typically, short range (a few kilometers), electro-optical EDM instruments with visible or near infrared continuous radiation are used in engineering surveys, though some long range (up to 60 km) electro-optical or microwave instruments are also available.

(1) The accuracy (standard deviation) of EDM instruments may be expressed in a general form as:

$$\sigma = (a^2 + b^2S^2)^{0.5} \qquad (9\text{-}2)$$

where "a" contains errors of the phase measurement and calibration errors of the so-called zero correction (additive constant of the instrument and of the reflector), while the value of "b" represents a scale error due to the aforementioned uncertainties in the determination of the refractive index and errors in the calibration of the modulation frequency. Typically, the value of "a" ranges from 3 mm to 5 mm. In the highest precision EDM instruments, such as the Kern ME5000, Geomensor CR234 (Com-Rad, U.K.), and Tellurometer MA200 (Tellumat, U.K.), a = 0.2 mm to 0.5 mm thanks to a high modulation frequency and high resolution of the phase measurements in those instruments.

(2) One recently developed engineering survey instrument is Leica (Wild) DI2002 which offers a

standard deviation of 1 mm over short distances. Over distances longer than a few hundred meters, however, the prevailing error in all EDM instruments is due to the difficulty in determining the refractive index. Therefore, all EDM measurements must be corrected for the actual refractive index of air along the measured distance. An error of 1˚C or an error of 3 mb in barometric pressure causes a 1 ppm (part per million or 1 $mm \cdot km^{-1}$) error of the measured distance. An extremely careful measurement of the atmospheric conditions at several points along the optical path must be performed with well calibrated thermometers and barometers in order to achieve the 1-ppm accuracy. If the meteorological conditions are measured only at the instrument station (usual practice), then errors of a few parts per million may occur, particularly in diversified topographic conditions. In order to achieve the accuracy better than 1 ppm, one has to either measure the meteorological conditions every few hundred meters (200 m - 300 m) along the optical path or to use EDM instruments with a dual frequency radiation source.

(3) Only a few units of a dual frequency instrument (Terrameter LDM2 by Terra Technology) are available around the world. They are bulky and capricious in use but one may achieve with them a standard deviation of ±0.1 mm ±0.1 ppm. Due to a small demand, its production has been discontinued. Research in the development of new dual frequency instruments is in progress. In deformation measurements one may reduce somewhat the influence of refraction by "calibrating" the distance observations to the object targets by comparing the results with the "fixed" distances between stable stations of the reference network.

(4) Influence of relative humidity may be neglected when using common electro-optical EDM instruments in moderate climatic conditions. The negligence of humidity, however, may cause errors up to 2 ppm in extremely hot and humid conditions. Therefore, in the highest precision measurements, psychrometers with wet and dry thermometers should be used to determine the correction due to water vapor content. One should always use rigorous formulas to calculate the refractive index correction rather than diagrams or simplified calculation methods supplied by the manufacturers.

(5) All EDM instruments must be frequently calibrated for the zero correction and for scale (change in the modulation frequency). The zero correction usually significantly changes with time and may also be a function of the intensity of the reflected signal. In some older EDM instruments, the zero correction may demonstrate phase dependent cyclic changes. In engineering projects of high precision, the EDM instruments should be calibrated at least twice a year, or before and after each important project. The calibration must account for all combinations of EDM-reflector pairings since each reflector may also have a different additive constant correction. Additional errors, which are not included in the general error equation, may arise when reducing the results of the spatially measured distances to a reference plane depending on the accuracy of the reduction corrections.

(6) Recently, a few models of EDM instruments with a short pulse transmission and direct measurement of the propagation time have become available. These instruments, having a high energy transmitted signal, may be used without reflectors to measure short distances (up to 200 m) directly to walls or natural flat surfaces with an accuracy of about 10 m. Examples are the Pulsar 500 (Fennel, Germany) and the Leica (Wild) DIOR 3002.

d. Total stations and survey robots.

(1) Any electronic theodolite linked to an EDM instrument and to a computer creates a total surveying station which allows for a simultaneous measurement of the three basic positioning parameters, distance, horizontal direction, and vertical angle, from which relative horizontal and vertical positions of the observed points can be determined directly in the field. Several manufacturers of survey equipment produce integrated total stations in which the EDM and electronic angle measurement systems are incorporated into one compact instrument with common pointing optics. Different models of total stations vary in accuracy, range, sophistication of the automatic data collection, and possibilities for on-line data processing. One of the most recommended total stations for precision engineering surveys is the Leica (Wild) TC2002 which combines the precision of the aforementioned electronic theodolite, Leica (Wild) T2002, with the precision EDM instrument, Leica (Wild) DI2002, into one instrument with a coaxial optics for both the angle and distance measurements.

(2) For continuous or frequent monitoring of deformations, fully automatic monitoring systems based on computerized and motorized total stations have recently been developed. The first system was Georobot. The recent advanced systems include, for example, the Geodimeter 140 SMS (Slope Monitoring System) and the Leica (Wild) APS and Georobot III systems based on the motorized TM 3000 series of Leica (Wild) electronic theodolites linked together with any Leica (Wild) DI series of EDM. These can be programmed for sequential self-pointing to a set of prism targets at predetermined

time intervals, can measure distances and horizontal and vertical angles, and can transmit the data to the office computer via a telemetry link. Similar systems are being developed by other manufacturers of surveying equipment. The robotic systems have found many applications, particularly in monitoring high walls in open pit mining and in slope stability studies. Generally, the accuracy of direction measurements with the self-pointing computerized theodolites is less accurate (about 3 sec) than the measurements with manual pointing.

9-17. Leveling and Trigonometric Height Measurements

a. The old method of geometrical leveling with horizontal lines of sight (using spirit or compensated levels) is still the most reliable and accurate, though slow, surveying method. With high magnification leveling instruments, equipped with the parallel glass plate micrometer and with invar graduated rods, a standard deviation smaller than 0.1 mm per setup may be achieved in height difference determination as long as the balanced lines of sight do not exceed 20 m. In leveling over long distances (with a number of instrument setups) with the lines of sight not exceeding 30 m, a standard deviation of 1 mm per km may be achieved in flat terrain. The aforementioned influences of atmospheric refraction and earth curvature are minimized by balancing the lines of sight between the forward and backward leveling rods. A dangerous accumulation of refraction error, up to 15 mm for each 100-m difference in elevation, may take place along moderately inclined long routes due to unequal heights of the forward and backward horizontal lines above the terrain.

b. The recently developed Leica (Wild) NA2000 and NA3000 digital automatic leveling systems with height and distance readout from encoded leveling rods have considerably increased the speed of leveling (by about 30 percent) and decreased the number of personnel needed on the survey crew. However, some users of the digital level NA3000 complain that its compensating system demonstrates systematic deviations in windy weather and, therefore, cannot be classified as a high precision level unless some improvements are introduced by the manufacturer.

c. High precision electronic theodolites and EDM equipment allow for the replacement of geodetic leveling with more economical trigonometric height measurements. Using precision electronic theodolites for vertical angle measurements and any short-range EDM instrument, one may achieve an accuracy better than 1 mm in height difference determination between two targets 200 m apart. To minimize the atmospheric refraction effects, the measurements must be performed either reciprocally, with two theodolites simultaneously, or from an auxiliary station with equal distances to the two targets (similar methodology as in spirit leveling). The accuracy is practically independent of the height differences and, therefore, is especially more economical than conventional leveling in hilly terrain and in all situations where large height differences between survey stations are involved. Motorized trigonometric height traversing (reciprocal or with balanced lines of sight) with precision theodolites and with the lines of sight not exceeding 250 m can give a standard deviation smaller than 2 mm per km. With automatic data collection and on-line processing of the measurements, daily progress of up to 15 km may be achieved independent of the terrain configuration. The refraction error is still the major problem in further increasing the accuracy of leveling. Research in this area continues.

9-18. Use of GPS in Deformation Surveys

a. The satellite GPS offers several advantages over conventional terrestrial methods. Intervisibility between stations is unnecessary, thus allowing greater flexibility in the selection of station locations than in the terrestrial geodetic surveys. Measurements can be taken during night or day, under varying weather conditions, which makes GPS measurements economical. With the recently developed rapid static positioning techniques, the time for the measurements at each station is reduced to a few minutes. Reference EM 1110-1-1003.

b. Though already widely used in engineering and geoscience projects, GPS is still a new and not perfectly known technology from the point of view of its optimal use and understanding of the sources of errors. The accuracy of GPS is very often exaggerated by some authors who may not quite understand the difference between the short-term precision (repeatability) and actual accuracy of GPS.

c. The accuracy of GPS relative positioning depends on the distribution (positional geometry) of the observed satellites and on the quality of the observations. There are several sources of errors contaminating the GPS measurements. These errors can be categorized into:

(1) Signal propagation errors which include effects of tropospheric and ionospheric refraction,

(2) Receiver related errors which include multipath effects, variation in the antenna phase center, receiver

noise, bias in the coordinates of the station being held fixed in the data reduction process, etc., and

(3) Satellite related errors which include mainly orbital errors.

d. Different types of errors affect GPS relative positioning in different ways. Some of the errors may have a systematic effect on the measured baselines producing significant scale errors and rotations. Due to the changeable geometrical distribution of the satellites and the resulting changeable systematic effects of the observation errors, repeated GPS surveys for the purpose of monitoring deformations can also be significantly influenced (up to a few ppm) by scale and rotation errors which, if undetected, may contaminate the derived deformation parameters leading to a misinterpretation of the behavior of the deformable body. A particular attention to the systematic influences should be paid when a GPS network is established along the shore of a large body of water and measurements are performed in a hot and humid climate.

e. Experience with the use of GPS in various deformation studies indicates that, with the available technology (receivers) and the distribution of the satellites in 1990-1991, the accuracy of GPS relative positioning over areas of up to 50 km in diameter can be expressed in terms of the variance of the horizontal components of the GPS baselines, over a distance S, as

$$\sigma^2 = (3\ \text{mm})^2 + (10^{-6}\ S)^2$$

if the aforementioned systematic biases (rotations and change in scale of the network) are identified and eliminated through proper modeling at the stage of the deformation interpretation. The accuracies of vertical components of the baselines are, usually, 1.5 to 2.5 times less accurate than the horizontal components.

f. The solution for the systematic parameters may be obtained by either (1) combining the GPS surveys of some baselines (of a different orientation) with terrestrial surveys of a compatible or better accuracy or (2) establishing several points outside the deformable area (fiducial stations) which would serve as a “calibration network” or (3) combining (1) and (2). These aspects must be considered when designing GPS networks for any engineering project. In the first case, for all the terrestrial $\mathbf{l}_T$ observables and for GPS observables $\mathbf{l}_G$ one could write observation equations for epochs t_o and t_i in terms of the deformation model **Bc** (displacement function) in the form:

$$\mathbf{l}_T\ (t_i) + \mathbf{v}_T(t_i) = \mathbf{l}_T(t_o) + \mathbf{A}_T\mathbf{B}_\mathbf{l}\mathbf{c},$$

for all i, and

$$\mathbf{l}_G\ (t_i) + \mathbf{v}_G(t_i) = \mathbf{l}_G(t_o) + \mathbf{A}_G\mathbf{B}_\mathbf{l}\mathbf{c} + \mathbf{D}d\eta$$

for all i, (9-3)

where $d\eta$ is the vector of changes in scale and rotation parameters between the epochs t_o and t_i, $\mathbf{B}_\mathbf{l}$ is the matrix constructed by superimposing matrices **B** for all the surveyed points and all the epochs, and **A** is the design matrix relating observables to the deformation model. The elements of the vectors **c** and $d\eta$ are estimated using the least squares method and they are statistically tested for their significance.

g. The influence of systematic errors in measurements over short distances (up to a few hundred meters) is usually negligible and the horizontal components of the GPS baselines can be determined with standard deviations of 3 mm or even smaller. Recent improvements to the software for the GPS data processing allow for an almost real time determination of changes in the positions of GPS stations.

h. In 1991, USACE developed a fully automated system for high-precision deformation surveys with GPS. It was designed particularly for dam monitoring. In the continuous deformation monitoring system (CDMS), GPS antennas are located over points to be monitored on the structures. At least two other GPS antennas must be located over reference points that are considered stable. The GPS antennas are connected to computers using a telemetry link. A prototype system used 10-channel Trimble 4000SL and Trimvec postprocessing software. An operator can access the onsite computer network through a remote hook-up in the office. In 1989 the system was installed at the Dworshak Dam on the Clearwater River near Orofino, ID. The results show that CDMS can give accuracies of 3 mm both horizontally and vertically over a 300-m baseline. One has to be aware, however, that although GPS does not require the intervisibility between the observing stations it requires an unobstructed view to the satellites which limits the use of GPS only to reasonably open areas. One should also remember that there might be some yet undiscovered sources of errors (e.g., effects of high voltage power lines) in GPS measurements. GPS certainly revolutionizes the geodetic surveys but still more research on its optimal use and on sources of errors in deformation surveys is needed. See also EM 1110-1-1003 for details on this system.

9-19. Photogrammetric Techniques

a. If an object is photographed from two or more survey points of known relative positions (known coordinates) with a known relative orientation of the camera(s), relative positions of any identifiable object points can be determined from the geometrical relationship between the intersecting optical rays which connect the image and object points. If the relative positions and orientation of the camera are unknown, some control points on the object must be first positioned using other surveying techniques. Aerial photogrammetry has been extensively used in determining ground movements in ground subsidence studies in mining areas, and terrestrial photogrammetry has been used in monitoring of engineering structures. The main advantages of using photogrammetry are: the reduced time of field work; simultaneous provision of 3-D coordinates; and, in principle, an unlimited number of points can be monitored. The accuracy of photogrammetric point determination has been much improved in the past decade, which makes it attractive for high precision deformation measurements.

b. Special cameras with minimized optical and film distortions must be used in precision photogrammetry. Cameras combined with theodolites (phototheodolites), for instance the Wild P-30 model, or stereocameras (two cameras mounted on a bar of known length) have found many applications in terrestrial engineering surveys including mapping and volume determination of underground excavations and profiling of tunnels. The accuracy of photogrammetric positioning with special cameras depends mainly on the accuracy of the determination of the image coordinates and the scale of the photographs. The image coordinates may, typically, be determined with an accuracy of about 10 µm, though 3 µm is achievable. The photo scale may be approximately expressed as f/s, where f is the focal length of the objective lens and s is the distance of the camera from the object. Using a camera with, for instance, f = 100 mm at a distance s = 100 m, with the accuracy of the image coordinates of 10 µm, the coordinates of the object points can be determined with the accuracy of 10 mm. Special large-format cameras with long focal length are used in close-range industrial applications of high precision. For instance, the model CRC-1 (Geodetic Services, Inc., U.S.A.) camera with f = 240 mm, can give sub-millimeter accuracy in "mapping" objects up to a few tens of meters away. Recently, solid-state cameras with charge couple device (CCD) sensors have become available for close-range photogrammetry in static as well as in dynamic applications. With the new developments in CCD cameras and digital image processing techniques, continuous monitoring with real-time photogrammetry becomes possible. Further development in this area is in progress.

9-20. Alignment Measurements

a. Alignment surveys cover an extremely wide spectrum of engineering applications from the tooling industry, through measurements of amplitude of vibrations of engineering structures, to deformation monitoring of nuclear accelerometers several kilometers long. Each application may require different specialized equipment.

b. The methods used in practice may be classified according to the method of establishing the reference line, that is: (1) mechanical methods in which stretched wire (steel, nylon, etc.) establishes the reference line, (2) direct optical method (called also collimation method), in which the optical line of sight or a laser beam "marks" the line, and (3) diffraction method in which the reference line is created by projecting a pattern of diffraction slits.

c. All the above methods except mechanical are affected by atmospheric refraction, as expressed by Equation 9-1. Therefore, in measurements requiring high accuracy, the alignment must be repeated several times in different environmental conditions.

d. The mechanical methods with tensioned wires as the reference lines have found many applications including dam deformation surveys. This is due to their simplicity, high accuracy, and easy adaptation to continuous monitoring of structural deformations using inductive sensors over distances up to a few hundred meters. Accuracies of 0.1 mm are achievable.

e. The direct optical method utilizes either an optical telescope and movable targets with micrometric sliding devices or a collimated (projected through the telescope) laser beam and movable photocentering targets. Besides the aforementioned influence of atmospheric refraction, pointing and focusing are the main sources of error when using optical telescopes. The pointing error with properly designed targets varies from 15"/M at night in calm atmospheric conditions to 60"/M in daylight with average turbulent conditions, where M is the magnification of the telescope.

f. Special aligning telescopes with large magnification (up to 100x) are available from, among others, Fennel-Cassell (Germany) and Zeiss-Jena (Germany). Aligning telescopes for the tooling industry and machinery alignment are available in North America from

Cubic Precision. When the optical line of sight is replaced by a collimated laser beam, then the accuracy of pointing may be considerably improved if special self-centering laser detectors, with a time integration of the laser beam energy, are used. The use of laser allows for automation of the alignment procedure and for continuous data acquisition. When using the laser beam directly as the reference line, however, attention must be paid to the stability of the laser cavity. A directional drift of the laser beam as high as 4"/°C may occur due to thermal effects on the laser cavity. This effect is decreased by a factor of M when projecting the laser through a telescope.

g. In diffraction alignment methods, a pinhole source of monochromatic (laser) light, the center of a plate with diffraction slits, and the center of an optical or photoelectric sensor are the three basic points of the alignment line. If two of the three points are fixed in their position, then the third may be aligned by centering the reticle on the interference pattern created by the diffraction grating. It should be pointed out that movements of the laser and of its output do not influence the accuracy of this method of alignment because the laser serves only as a source of monochromatic light placed behind the pinhole and not as the reference line. Therefore, any kind of laser may be employed in this method, even the simplest and least expensive ones, as long as the output power requirements are satisfied. Various patterns of diffraction slits are used in practice. The highest accuracy and the longest range are obtained with the so-called Fresnel zone plates which act as focusing lenses. For instance, rectangular Fresnel zone plates with an electro-optical centering device were used in alignment and deformation measurements of a 3-km-long nuclear accelerator giving relative accuracy (in a vacuum) of 10^{-7} of the distance. In the open atmosphere, the thermal turbulence of air seems to have a smaller effect when using the Fresnel zone plates than in the case of direct optical alignment. The laser diffraction alignment methods have successfully been applied in monitoring both straight and curved (arch) dams using self-centering targets with automatic data recording.

9-21. Measurement of Extension (Change in Distance) and Strain

a. Types of extensometers. Various types of instruments, mainly mechanical and electro-mechanical, are used to measure changes in distance in order to determine compaction or upheaval of soil, convergence of walls in engineering structures and underground excavations, strain in rocks and in man-made materials, separation between rock layers around driven tunnels, slope stability, and movements of structures with respect to the foundation rocks. Depending on its particular application, the same instrument may be named an extensometer, strainmeter, convergencemeter, or fissuremeter.

(1) The various instruments differ from each other by the method of linking together the points between which the change in the distance is to be determined and the kind of sensor employed to measure the change. The links in most instruments are mechanical, such as wires, rods, or tubes. The sensors usually are mechanical, such as calipers or dial gauges. In order to adapt them to automatic and continuous data recording, electric transducers can be employed using, for instance, linear potentiometers, differential transformers, and self-inductance resonant circuits. In general, when choosing the kind of transducer for automatic data acquisition, one should consult with an electronics specialist on which kind would best suit the purpose of the measurements in the given environmental conditions.

(2) One should point out that the precision EDM instruments, as described earlier with their accuracy of 0.3 mm over short distances, may also be used as extensometers particularly when the distances involved are several tens of meters long.

(3) If an extensometer is installed in the material with a homogeneous strain field, then the measured change Δl of the distance l gives directly the strain component $\varepsilon = \Delta l/l$ in the direction of the measurements. To determine the total strain tensor in a plane (two normal strains and one shearing), a minimum of three extensometers must be installed in three different directions.

b. Wire and tape extensometers. Maintaining a constant tension throughout the use of the wire or tape extensometer is very important. In some portable extensometers, the constant tensioning weight has been replaced by precision tensioning springs. One should be careful because there are several models of spring tensioned extensometers on the market which do not provide any means of tension calibration. As the spring ages, these instruments may indicate false expansion results unless they are carefully calibrated on a baseline of constant length, before and after each measuring campaign.

(1) Among the most precise wire extensometers are the Kern Distometer (discontinued production) and the CERN Distinvar (Switzerland). Both instruments use invar wires and special constant tensioning devices which, if properly calibrated and used, can give accuracies of

0.05 mm or better in measurements of changes of distances over lengths from about 1 m to about 20 m. Invar is a capricious alloy and must be handled very carefully to avoid sudden changes in the length of the wire. When only small changes in temperature are expected or a smaller precision (0.1 mm to 1 mm) is required, then steel wires or steel tapes are more comfortable to use.

(2) Special high precision strainmeters of a short length (up to a few decimeters) are available for strain measurements in structural material and in homogeneous rocks. An example is a vibrating wire strain gauge available from Rocktest (formerly Irad Gage). The instrument employs a 150-mm steel wire in which the changeable resonant frequency is measured. An accuracy of one microstrain (10^{-6}) is claimed in the strain measurements which corresponds to 0.15 µm relative displacements of points over a distance of 150 mm.

c. Rod, tube, and torpedo extensometers. Steel, invar, aluminum, or fiberglass rods of various lengths, together with sensors of their movements, may be used depending on the application. Multiple point measurements in boreholes or in trenches may be made using either a parallel arrangement of rods anchored at different distances from the sensing head, or a string (in series) arrangement with intermediate sensors of the relative movements of the rods.

(1) A typical accuracy of 0.1 mm to 0.5 mm may be achieved up to a total length of 200 m (usually in segments of 3 m to 6 m). The actual accuracy depends on the temperature corrections and on the quality of the installation of the extensometer. When installing rods in plastic conduit (usually when installing in boreholes), the friction between the rod and the conduit may significantly distort the extensometer indications if the length of the extensometer exceeds a few tens of meters. The dial indicator readout may be replaced by potentiometric or other transducers with digital readout systems. Telescopic tubes may replace rods in some simple applications, for instance, in measurements of convergence between the roof and floor of openings in underground mining.

(2) Several models of torpedo borehole extensometers and sliding micrometers are available from different companies producing geotechnical instrumentation. For example, Extensofor (Telemac, France) consists of a 28-mm-diameter torpedo 1.55 m long with an inductance sensor at each end. Reference rings on the casing are spaced within the length of the torpedo. The sensors and reference rings form the inductance oscillating circuits. The torpedo is lowered in the borehole and stopped between the successive rings recording changes in distances between the pairs of rings with a claimed accuracy of 0.1 mm. Boreholes up to several hundreds of meters long can be scanned.

d. Interferometric measurements of linear displacements. Various kinds of interferometers using lasers as a source of monochromatic radiation are becoming common tools in precision displacement measurements. A linear resolution of 0.01 µm, or even better, is achievable. One has to remember, however, that interferometric distance measurements are affected by atmospheric refractivity in the same way as all EDM systems. Therefore, even if temperature and barometric pressure corrections are applied, the practical accuracy limit is about 10^{-6} S (equivalent to 1 µm per meter). Thermal turbulence of air limits the range of interferometric measurements in the open atmosphere to about 60 m. The Hewlett Packard Model 5526B laser interferometer has found many industrial and laboratory applications in the measurement of small displacements and the calibration of surveying instruments.

e. Use of optical fibre sensors. A new interesting development in the measurements of extensions and changes in crack width employs a fully automatic extensometer which utilizes the principle of electro-optical distance measurements within fibre optic conduits. The changes in length of the fibre optic sensors are sensed electro-optically and they are computer controlled .

9-22. Tilt and Inclination Measurements

a. Methods of tilt measurements.

(1) The measurement of tilt is usually understood as the determination of a deviation from the horizontal plane, while inclination is interpreted as a deviation from the vertical. Thus the same instrument that measures tilt at a point can be called either a tiltmeter or an inclinometer, depending on the interpretation of the results.

(2) As discussed previously, geodetic leveling techniques can achieve an accuracy of 0.1 mm over a distance of 20 m, which would be equivalent to about 1.0" of angular tilt. This accuracy is more than sufficient in most engineering deformation measurements. Whenever a higher accuracy or continuous or very frequent collection of information on the tilt changes is necessary, however, various in situ instruments are used, such as (a) engineering tiltmeters and inclinometers; (b) suspended and inverted plumb lines; and (c) hydrostatic levels. In addition, some other specialized instruments such as

mercury/laser levels have been developed but are not commonly used in practice and, therefore, are not reviewed in this section.

b. Tiltmeters and inclinometers. There are many reasonably priced models of various liquid, electrolytic, vibrating wire, and pendulum type tiltmeters that satisfy most of the needs of engineering surveys. Particularly popular are servo-accelerometer tiltmeters with a small horizontal pendulum. They offer ruggedness, durability, and low temperature operation. The output signal (volts) is proportional to the sine of the angle of tilt. The typical output voltage range for tiltmeters is ±5 V, which corresponds to the maximum range of the tilt. Thus the angular resolution depends on the tilt range of the selected model of tiltmeter and the resolution of the voltmeter (typically 1 mV). There are many factors affecting the accuracy of tilt sensing. A temperature change produces dimensional changes of the mechanical components, changes in the viscosity of the liquid in the electrolytic tiltmeters, and changes of the damping oil in the pendulum tiltmeters. Drifts of tilt indications and fluctuations of the readout may also occur. Therefore, thorough testing and calibration are required even when the accuracy requirement is not very high.

(1) Tiltmeters have a wide range of applications. A series of tiltmeters if arranged along a terrain profile may replace geodetic leveling in the determination of ground subsidence. Similarly, deformation profiles of tall structures may be determined by placing a series of tiltmeters at different levels of the structure.

(2) In geomechanical engineering, the most popular application of tiltmeters is in slope stability studies and in monitoring embankment dams using the torpedo (scanning) type borehole inclinometers (usually the servo-accelerometer type tiltmeters). The biaxial inclinometers are used to scan boreholes drilled to the depth of an expected stable strata in the slope. By lowering the inclinometer on a cable with marked intervals and taking readings of the inclinometer at those intervals, a full profile of the borehole and its changes may be determined through repeated surveys. Usually the servo-accelerometer inclinometers are used with various ranges of inclination measurements, for instance, ±6˚, ±54˚, or even ±90˚. If a 40-m-deep borehole is measured every 50 cm with an inclinometer of only 100" accuracy, then the linear lateral displacement of the collar of the borehole could be determined with an accuracy of 2 mm. A fully automatic (computerized) borehole scanning inclinometer system with a telemetric data acquisition has been designed at the University of New Brunswick for monitoring slope stability at the Syncrude Canada tar sands mining operation.

c. Suspended and inverted plumb lines. Two kinds of mechanical plumbing are used in controlling the stability of vertical structures: (1) suspended plumb lines and (2) floating plumb lines, also called inverted or reversed plumb lines. Inverted plumb lines have an advantage over suspended plumb lines in the possibility of monitoring absolute displacements of structures with respect to deeply anchored points in the foundation rocks which may be considered as stable. In the case of power dams, the depth of the anchors must be 50 m or even more below the foundation in order to obtain absolute displacements of the surface points. If invar wire is used for the inverted plumb line, vertical movements of the investigated structure with respect to the bedrock can also be determined. Caution must be used in installing plumb lines. If the plumb line is installed outside the dam, a vertical pipe of a proper inner diameter should be used to protect the wire from the wind. The main concern with floating plumb lines is to ensure verticality of the boreholes so that the wire of the plumb line has freedom of motion. The tank containing the float is generally filled with water to which some antifreeze can be added. The volume of the float should be such as to exert sufficient tension on the wire. It should also be noted, however, that in a float tank thermal convection displacements may easily develop in consequence of thermal gradients which may affect measurements to a considerable extent. Hence in some cases, the whole tank should be thermally insulated.

(1) Several types of recording devices that measure displacements of structural points with respect to the vertical plumb lines are produced by different companies. The simplest are mechanical or electromechanical micrometers. With these, the plumb wire can be positioned with respect to reference lines of a recording (coordinating) table to an accuracy of ±0.1 mm or better. Travelling microscopes may give the same accuracy. Automatic sensing and recording is possible, for instance, with a Telecoordinator (Huggenberger, Switzerland) and with a Telependulum (Telemac, France). An interesting Automated Vision System has been developed by Spectron Engineering. The system uses CCD video cameras to image the plumb line with a resolution of about 3 µm over a range of 75 mm. Several plumb lines at the Glen Canyon dam and at the Monticello dam in California have used this system.

(2) Two sources of error which may sometimes be underestimated by users are: the influence of air currents

and the spiral shape of wires. To reduce the influence of the air pressure, the plumb line should be protected within a pipe (e.g., a PVC tube) with openings only at the reading tables.

d. Optical plummets. Several surveying instrument companies produce high precision optical plummets: Leica (Wild) ZL (zenith) and NL (nadir) plummets which offer the accuracy of 1/200,000. Both can be equipped with laser. The atmospheric refraction is the major source of errors.

e. Hydrostatic leveling. If two connected containers are partially filled with a liquid, then the heights h_1 and h_2 of the liquid in the containers are related through the hydrostatic equation

$$h_1 + P_1 / (g_1 \ r_1) = h_2 + P_2 / (g_2 \ r_2) = \text{const.}$$

where P is the barometric pressure, g is gravity, and r is the density of the liquid which is a function of temperature.

(1) The above relationship has been employed in hydrostatic leveling. The ELWAAG 001 (Bayernwerke, Germany) is a fully automatic instrument with a travelling (by means of an electric stepping motor) sensor pin which closes the electric circuit upon touching the surface of the liquid.

(2) Hydrostatic leveling is frequently used in the form of a network of permanently installed instruments filled with a liquid and connected by hose-pipes to monitor change in height differences of large structures. The height differences of the liquid levels are automatically recorded. The accuracy ranges from 0.1 mm to 0.01 mm over a few tens of meters depending on the types of instruments. The main factor limiting the survey accuracy is the temperature effect. To reduce this effect the instrument must either be installed in a place with small temperature variations, or the temperature along the pipes must be measured and corrections applied, or a double liquid (e.g., water and mercury) is employed to derive the correction for this effect. For the highest accuracy, water of a constant temperature is pumped into the system just before taking the readings. The instruments with direct measurement of the liquid levels are limited in the vertical range by the height of the containers. This problem may be overcome if liquid pressures are measured instead of the changes in elevation of the water levels. Pneumatic pressure cells or pressure transducer cells may be used.

9-23. Concluding Remarks on Monitoring Techniques

This brief review of basic monitoring techniques indicates that, from the point of view of the achievable instrumental accuracy, the distinction between geodetic and geotechnical techniques does not apply any more. With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any, practically needed, instrumental resolution and precision, full automation, and virtually real-time data processing. Remotely controlled telemetric data acquisition systems, working continuously for several months without recharging the batteries in temperatures down to -40˚C, are available and their cost is reasonable. Thus, the array of different types of instruments available for deformation studies has significantly broadened within the last few years. This creates a new challenge for the designers of the monitoring surveys: what instruments to choose, where to locate them, and how to combine them into one integrated monitoring scheme in which the geodetic and geotechnical/structural measurements would optimally complement each other.

a. As far as the actual accuracy of deformation surveys is concerned, the main limiting factors are not the instrument precision but the environmental influences and ignorance of the users, namely:

- The aforementioned atmospheric refraction.
- Thermal influences affecting the mechanical, electronic, and optical components of the instruments (in any type of instrumentation) as well as the stability of survey stations.
- Local instability of the observation stations (improper monumentation of survey stations and improper installation of the in situ instrumentation).
- Lack of or improper calibration of the instruments.
- Lack of understanding by the users of the sources of errors and of the proper use of the collected observations.

b. The problem of calibration is very often underestimated in practice not only by the users but also by the manufacturers. In long-term measurements, the instrument repeatability (precision) may be affected by aging of the electronic and mechanical components resulting in a drift of the instrument readout. Of particular concern are geotechnical instruments for which the users, in general, do not have sufficient facilities and adequate knowledge for their calibration. The permanently installed instruments are very often left in situ for several years without checking the quality of their performance.

c. The last aspect, the lack of understanding of the sources of errors affecting various types of measurements and the proper data handling is, perhaps, the most dangerous and, unfortunately, the frequent case in measurements of deformations in North America. The measurements, and particularly processing of the geodetic surveys, are usually in the hands of self-proclaimed "surveyors" at the technician level or even without any formal education. In this case, even the most technologically advanced instrumentation will not supply the expected information.

9-24. Design of Monitoring Schemes

a. When designing a monitoring network one has to remember that the main purpose of the monitoring surveys is:

- To check whether the behavior of the investigated object and its environment follows the predicted pattern so that any unpredicted deformations could be detected at an early stage.

- In the case of any abnormal behavior, to give an account, as accurately as possible, of the actual deformation status which could be used for the determination of the causative factors which trigger the deformation.

b. In the first case, the design of the monitoring scheme must include stations at the points where maximum deformations have been predicted plus a few observables at the points which, depending on previous experience, could signal any potential unpredictable behavior, particularly at the interface between the monitored structure and the surrounding material. The amount of the expected deformations may be predicted using either deterministic modeling (using, for instance, the finite or boundary element methods), or empirical (statistical) prediction models. Once any abnormal deformations are noticed, additional observables have to be added at the locations which would be indicated by the preliminary analysis of the monitoring surveys as being the most sensitive for the identification of causative factors. Some redundant monitoring instruments and points are absolutely necessary for checking the reliability of the measurements, especially in some critical parts of the structure. One should bear in mind that any monitoring instruments, even if they have been installed permanently, cannot rule out defects and failures. Thus, any monitoring system should be sufficiently redundant. By redundancy one means keeping parallel but separate sets of instruments and, in addition, facilities for evaluating data by double-checking, using alternative measurement methods. Examples would be the determination of relative displacements using alignment surveys versus displacements obtained from a geodetic monitoring network, the measurement of tilts with tiltmeters versus geodetic leveling, etc. Thus, a properly designed monitoring scheme should have a sufficient redundancy of measurements using different measuring techniques and such geometry of the scheme that self-checking, through geometrical closures of loops of measurements, would be possible. One should stress that a poorly designed monitoring survey is a waste of effort and money and may lead to a dangerous misinterpretation.

c. The accuracy (at the 95 percent probability level) of the monitoring measurements should be equal to at least 0.25 of the predicted value of the maximum deformations for the given span of time between the repeated measurements. However, once any abnormal deformations are noticed, there is no limit, other than economic, for the maximum possible accuracy required. The higher the accuracy of the measurements, the easier it will be to determine the mechanism of the unpredicted deformations. Thus, the monitoring schemes may require frequent updating and upgrading of the initial design over the duration of the monitoring project.

d. Generally, the design of a monitoring scheme includes, among many other aspects, the following tasks:

- Identification of the parameters to be observed.

- Selection of locations for the monitoring stations (both object and reference points if applicable).

- Determination (preanalysis) of the required accuracy and of the measuring range.

- Determination of the required frequency of repeated observations.

- Selection of the types of instruments and sensors to be used (various alternatives).

- Design of testing and calibration facilities.

- Design of the data management system.

- Preparation of a scenario for instrumentation failure (design of redundancy).

- Cost analysis and final decision on the selected monitoring scheme.

e. Since each deformable object may require different parameters to be observed, different instrumentation, different accuracies, and different frequencies of the observation, detailed specifications will significantly vary not only from one type of object to another, but also for the same type of object depending on the local surrounding conditions. Therefore, the following brief discussion on the first four tasks as listed above is very general and has been limited to typical conditions of concrete and embankment dams only.

9-25. Basic Considerations in Designing Monitoring Schemes for Large Dams

a. General deformation behavior of dams. Any dam is subjected to external and internal loads that cause deformation and permeability of the structure and its foundation. Deformation and seepage are clearly a function of such loads. Any sign of abnormal dam behavior could signal a threat to dam safety. In view of the difference between concrete and embankment dam behavior, a monitoring scheme cannot be organized in the same way for both types. In concrete dams, monitoring is essentially a matter of observing behavioral trends in both elastic and plastic deformation. The work consists of comparing measured deformation to the predicted normal behavior, assessed through analysis or some other method. In embankment dams on the other hand, permanent deformation trends should be closely monitored for any sign of abnormality.

b. Identification of parameters to be observed in concrete dams.

(1) Absolute horizontal and vertical displacements. These measurements are particularly intended to determine the small displacements of points representative of the behavior of the dam, its foundation, and abutments with respect to some stable frame. Geodetic surveys are often used for this purpose. The "absolute values" can be obtained only if the reference points are stable. Here, the GPS technique helps in establishing stations far enough from the dam to be outside of the deformation zone of the reservoir. In order to efficiently check their stability the number of reference points must be not less than 3, preferably 4, for vertical control, and 4, preferably 6, for horizontal, and they should be connected together by observables, with as much redundancy as possible. Horizontal displacements in a critical direction, usually perpendicular to the axis of dam, can be surveyed with alignment techniques if the reference points of the alignment survey are stable or their movements can be determined by other techniques, for instance, by inverted plumb lines with a stable anchor point, or by geodetic methods. Vertical absolute displacements can be determined by geodetic leveling with respect to deeply anchored vertical borehole extensometers or to deep benchmarks located near the dam. Long leveling lines connecting the dam with benchmarks located several kilometers outside the deformation zone are not recommended due to the accumulation of errors.

(2) Relative movements. Deflections (inclinations) of a dam are usually measured by direct or inverted plumb lines. With reference to a horizontal line along the axis of the dam, different alignment methods are used in different levels of galleries to determine the relative movements between the blocks. Extensometers have now become important instruments for measuring differential foundation movements. A combination of geodetic leveling with suspended invar wires equipped with short reading scales at different levels of the dam and connected to borehole extensometers can supply all the needed information on the relative vertical movements as well as on the absolute vertical displacements and relative tilts.

(3) Foundation subsidence and tilts. They are measured with geodetic leveling, hydrostatic leveling, and tiltmeters. The last two are usually permanently installed in galleries.

(4) Strain measurements. Strain gauges are preferably embedded in the concrete during construction, installed on the faces of the dam after completion, or even embedded in foundation boreholes.

(5) Temperature. Temperature measurements should provide information on the thermal state of the concrete, water temperature at various levels, and atmospheric temperature. Temperature in the concrete is usually measured by telethermometers (thermistors, thermocouples, bi-metal thermometers) installed in the dam body.

(6) Uplift and leakage measurements. These measurements are generally carried out by nonspecific instruments. More elaborate devices may, however, be required to measure hydraulic pressure inside the rock. For

leakage measurements, it is important to combine an effective drainage system with these instruments.

(7) Joint measurements. Measurements are justified only in the case of joints separating two unsealed structures or to check grouting in dome or arch-gravity dams. Cracks are measured by the same methods, the instruments being installed on the surface.

(8) Water level measurements. Water level in a reservoir is one of the most important acting loads to a dam. For physical interpretation its measurement should coincide in time with the measurements of other deformation quantities.

c. Identification of parameters to be observed in embankment dams.

(1) Horizontal displacements. Horizontal displacements of the crest and other important points of embankment (berms, etc.) can be measured with geodetic methods and alignment. The comments made for concrete dams are also valid here. It is also possible to detect relative horizontal displacements of points inside the embankment by means of inclinometers.

(2) Groundwater and pore water pressures. Groundwater and pore pressures are very significant in monitoring earth dams. The pattern of seepage and pore water pressure, especially in the foundation and the impervious core, has a significant impact on the normal behavior of embankment dams. Since pore water pressures should not exceed design values, they must be carefully monitored, possibly with pressure cells. The greater the number of measurement profiles and the number of cells per profile, the more useful the data obtained will be.

(3) Settlements. For those occurring in accessible places, geodetic or hydrostatic leveling is customarily used to determine the settlements. The settlements of the foundation, or of interior structural parts which are not accessible (core, foundation contact), are detected through settlement gauges. The settlements of individual layers of the embankment should be monitored. This can be done through settlement gauges installed in the different layers.

(4) Total pressure measurements. It is sometimes necessary to check the total pressure inside the embankment or between the embankment and the foundation or adjacent structures.

(5) Water level measurements. Water level in the reservoir is the most important load on an earth dam, causing horizontal movements and seepage. Its measurement should coincide in time with measurement of the deformations and seepage.

d. Location of monitoring instruments. In addition to the general guidelines given above, for gravity dams, each block should have at least one point. Tilts of the foundation should be measured at the center for small structures, and at not less than three points for larger structures.

(1) For multiple-arch and buttress dams, monitoring points should be located at the head and downstream toe of each buttress. In the case of massive buttresses and large arches, special attention should be paid to the foundations of the buttresses. If the buttresses are tranversed by construction joints, the behavior of joints should be observed.

(2) For arch-gravity dams and thick arch dams, absolute displacements of dam toe and abutments are critical. For small structures, the deformation of the central block is monitored. However, for large structures the measurement of deformations in each block is required.

(3) For thin arch dams, crest displacements in the horizontal and vertical are required. Special attention should be given to central cantilever, abutments, and abutment rock.

e. Accuracy requirements.

(1) No commonly accepted standards of accuracy requirements exist. As aforementioned, the accuracy at 95 percent probability should be equal to at least 0.25 of the maximum expected deformation regular behavior, and as high as possible for a discovered irregularity.

(2) For concrete dams, the accuracy for monitoring both horizontal and vertical displacements should typically range between 5 and 10 mm. For earth-rockfill dams, the accuracy should be about 20-30 mm for horizontal displacements and 10 mm for settlements during construction and operation. During operation, accuracies of 10-20 mm are normally required.

f. Frequency of measurements. The frequency of periodic measurements depends on the age of the structure and type of the monitoring system. If a fully automatic data acquisition system is used, the frequency of measurements does not impose any problem because the data can be decoded at any preprogrammed time intervals without any logistic difficulties and, practically, at no difference in the cost of the monitoring process. However, in most cases, the fully automated systems are not yet commonly used and the frequency of measurements of individual observables must be carefully designed to compromise between the actual need and the cost. ICOLD gives the following general guidelines:

(1) Before and during construction, it may be useful to carry out some geodetic and piezometric measurements of the abutments.

(2) All measurements should be made before the first filling is started (initial operation). The dates of the successive measurements will depend on the level the water has reached in the reservoir. The closer the water is to the top level, the shorter will be the interval between the measurements. For instance, one survey should be conducted when the water reaches one-fourth of the total height; another survey when the water reaches midheight; one survey every tenth of the total height for the third quarter; one survey every 2 m of variation for the fourth quarter. Moreover, the interval between two successive surveys should never exceed a month until filling is completed.

(3) During the operation of the structure, measurements should be more frequent in the years immediately following the first filling when active deformation is in progress. For instance, geodetic surveys (more labor intensive) can be carried out four times a year, and other geotechnical measurements can be made once every 1 to 2 weeks.

(4) After the structure is stable, which takes usually 5 to 10 years or more, the above frequencies can be reduced by half. Not only the frequencies of measurement, but also the number of instruments read can be reduced according to what is learned during the first years of operation.

9-26. Optimal Design of the Configuration and Accuracy of the Monitoring Schemes

a. The general guidelines and restrictions regarding the locations and accuracy of instrumentation, discussed above, still give room for choice (within the general guidelines) of the final positions of, at least, some observation stations and accuracy of the observations. This concerns mainly the geodetic monitoring reference networks, which may provide different solutions for the accuracy of the observed displacements depending on the selected configuration of the connecting surveys, location of the reference stations, and type of observables (e.g., distances as opposed to angles, or an optimal combination of both).

b. The optimum design of geodetic positioning networks has been the subject of intensive investigations and publications by many authors over the past two decades. The optimization of geodetic positioning networks is aimed at obtaining the optimum positions of the geodetic points with the optimum accuracy, reliability, and economy of the survey scheme taken as the design criteria. Design of deformation monitoring schemes is more complex and differs in many respects from the design of positioning networks. The design is aimed at obtaining optimum accuracies for the deformation parameters rather than for the coordinates of the monitoring stations using various types (geodetic and nongeodetic) observables with allowable configuration defects. The sensitivity of the monitoring scheme to detect deformations is introduced as more general than the accuracy design criterion. Very recently, a separability concept has been added to the design criteria (separation between different possible deformation models).

c. There are practically two distinct optimal design methods:

- Analytical.

- Computer simulation (“trial and error”).

Both methods usually involve an iteration process. The difference between them is that the former does not require human intervention and provides, mathematically, the optimum results, while the computer simulation method (CSM) provides acceptable results but they are not necessarily optimal. The CSM requires the experience of the designer but it can solve all the design problems while the analytical methods have been limited to some particular solutions only.

d. Recently, a CSM has been developed which does not require any human intervention during a fully automatic computational process from the moment of imputting the initial data through the iterative step-by-step upgrading of the design to the final design output. Although the underlying theory for the design of geodetic networks has been developed quite extensively, its full power of practical application has not been demonstrated in any real-life examples. An efficient algorithm has not existed until very recently. The major problem in this area was the inability of solving nonlinear matrix equations involved in the network design. This problem has recently been solved at the University of New Brunswick

(UNB) by developing a multiobjective analytical design methodology which allows for a fully analytical, multi-objective optimal design (optimal accuracy and sensitivity, optimal reliability, and optimal economy) of integrated deformation monitoring schemes with geodetic and geo-technical instrumentation. The method allows for a simultaneous solution for the optimal configuration and accuracy of the monitoring scheme according to the given criteria and restrictions concerning the locations of some observation stations and required accuracy of the deformation parameters.

9-27. Analysis of Deformation Surveys

Even the most precise monitoring surveys will not fully serve their purpose if they are not properly evaluated and utilized in a global integrated analysis as a cooperative interdisciplinary effort. The analysis of deformation surveys includes:

- Geometrical analysis which describes the geometrical status of the deformable body, its change in shape and dimensions, as well as rigid body movements (translations and rotations) of the whole deformable body with respect to a stable reference frame or of a block of the body with respect to other blocks, and

- Physical interpretation which consists of: (a) a statistical (stochastic) method, which analyzes through a regression analysis the correlations between observed deformations and observed loads (external and internal causes producing the deformation), and (b) a deterministic method, which utilizes information on the loads, properties of the materials, and physical laws governing the stress-strain relationship which describes the state of internal stresses and the relationship between the causative effects (loads) and deformations.

a. Once the load-deformation relationship is established, the results of the physical interpretation may be used for the development of prediction models. Through a comparison of predicted deformation with the results of the geometrical analysis of the actual deformations, a better understanding of the mechanism of the deformations is achieved. On the other hand, the prediction models supply information on the expected deformation, facilitating the design of the monitoring scheme as well as the selection of the deformation model in the geometrical analysis. Thus, the expression integrated analysis means a determination of the deformation by combining all types of measurements, geodetic and geotechnical, even if scattered in time and space, in the simultaneous geometrical analysis of the deformation, comparing it with the prediction models, enhancing the prediction models which, in turn, may be used in enhancing the monitoring scheme. The process is iteratively repeated until the mechanism of deformation is well understood and any discrepancies between the prediction models and actual deformations are properly explained.

b. Recently, the concept of global integration has been developed in which all three — the geometrical analysis of deformation and both methods of the physical interpretation — are combined into a simultaneous solution for all the parameters to be sought. The method still requires further elaboration, software development, and practical testing and, therefore, is not described in detail in this manual.

c. The deterministic and statistical modeling of deformations have been used in the analysis of dam deformations, at least in some countries, for many years. As aforementioned, the geometrical analysis has been done so far in a rather primitive way with geotechnical/structural engineers analyzing separately the geotechnical observation data and surveyors taking care of the geodetic survey observations. The geotechnical analyses have usually resulted only in a graphical display of temporal trends for individual observables and the geodetic analysis would result in a plot of displacements obtained from repeated surveys which, very often, would not be even properly adjusted and analyzed for the stability of the reference points. Over the past ten years, an intensive study by the FIG working group has resulted in the development of proper methods for the analysis of geodetic surveys and has led to the development of the so-called UNB Generalized Method of the geometrical deformation analysis which can combine any type of observations (geotechnical and geodetic) into one simultaneous analysis. The developed methodology for the geometrical analysis is given below followed by brief descriptions of the statistical and deterministic methods used in modeling the load-deformation relationship. Finally, the concept of the hybrid physical analysis in which the statistical modeling is combined with the deterministic method is described in more detail.

9-28. Geometrical Analysis of Deformation Surveys

a. Identification of unstable reference points. In most deformation studies, the information on absolute movements of object points with respect to some stable reference points is crucial. One problem which is frequently encountered in practice in the reference networks is the instability of the reference points. This may be caused either by wrong monumentation of the survey markers or by the points being located still too close to the deformation zone (wrong assumption in the design about the stability of the surrounding area). Any unstable reference points must be identified first before the absolute displacements of the object points are calculated. Otherwise, the calculated displacements of the object points and subsequent analysis and interpretation of the deformation of the structure may be significantly distorted. Given a situation where points A, B, C, and D are reference points used to monitor a number of object points on a structure; if point B has moved but this is not recognized and it is used with point A to identify the common datum for two survey campaigns, then all the object points and reference points C and D will show significant changes in their coordinates even when, in reality, they are stable.

(1) Over the past two decades several methods for the analysis of reference networks have been developed in various research centers within the activity of the aforementioned FIG Study Group. There are basically two schools of thought. One is based on the congruency test, and the other is based on defining a datum for the second epoch of measurements which is robust to unstable reference points. In the first case, a failure in the congruency test is followed by a search for the new congruency test which has a minimum statistic. The test statistics are calculated by removing points, one by one in turn, from the set of reference points until all the unstable points are identified. In the second case, a method has been developed at UNB which is based on a special similarity transformation which minimizes the first norm of the vector of displacements of the reference points. The approach can be performed easily for one-dimensional reference networks and by an iterative weighting scheme for multi-dimensional reference networks until all the components of the displacement vectors satisfy the condition: $\sum | d_i |$ = minimum. In each solution, the weights are iteratively changed to be $p_i = 1/d_i$. After the last iteration, the displacement vectors that exceed their error ellipses at 95 percent probability identify the unstable reference points. The displacements obtained from the iterative weighted transformation are, practically, datum independent; i.e., that whatever minimum constraints have been used in the least squares adjustment of the survey campaigns, the display of the transformed displacements will always be the same. Thus the obtained graphical display represents the actual deformation trend which is used later on in selecting the best fitting deformation model, as described below.

(2) Several software packages for geometrical analysis have been developed, for example, DEFNAN, PANDA, and LOCAL. Some of them (e.g., DEFNAN) are applicable not only to the identification of unstable reference points, but to the integrated analysis of any type of deformations (to be discussed below), while others are limited to the analysis of reference geodetic networks only.

b. UNB Generalized Method for geometrical deformation analysis. In order to be able to utilize any type of geodetic and geotechnical observations in a simultaneous deformation analysis, the UNB Generalized Method of the geometrical analysis has been developed. The method is applicable to any type of geometrical analysis, both in space and in time, including the earlier discussed detection of unstable reference points and the determination of strain components and relative rigid body motion within a deformable body. It allows utilization of different types of surveying data (conventional surveys and GPS measurements) and geotechnical/structural measurements. It can be applied to any configuration of the monitoring scheme as long as approximate coordinates of all the observation points are known. In practical applications, the approach consists of three basic processes:

- Identification of deformation models.
- Estimation of deformation parameters.
- Diagnostic checking of the models, and final selection of the “best” model.

A brief description of the approach is given below.

(1) The change in shape and dimensions of a 3-D deformable body is fully described if 6 strain components (3 normal and 3 shearing strains) and 3 differential rotations at every point of the body are determined. These deformation parameters can be calculated from the well-known strain-displacement relations if a displacement function representing the deformation of the object is known. Since, in practice, deformation surveys involve only discrete points, the displacement function must be approximated through some selected deformation model which fits the observed changes in coordinates (displacements), or any other types of observables, in the statistically best way. The displacement function may be determined, for example, through a polynomial approximation of the displacement field.

(2) The displacement function can be expressed in matrix form in terms of a deformation model **Bc** as:

$$\mathbf{d}(x,y,z,t\text{-}t_o) = (u,v,w)^T = \mathbf{B}(x,y,z,t\text{-}t_o)\ \mathbf{c} \qquad (9\text{-}4)$$

where **d** is the displacement of a point (x,y,z) at time t in respect to a reference time t_o; u,v,and w are components of the displacement function in the x-, y-, and z- directions, respectively, **B** is the deformation matrix with its elements being some selected base functions, and **c** is the vector of unknown coefficients (deformation parameters).

(3) For illustration, examples of typical deformation models (displacement functions) in 2-D analysis are given below.

(a) Single point displacement or a rigid body displacement of a group of points, say, block B with respect to block A. The deformation model is expressed in the form of the following displacement functions:

$$u_A = 0,\ v_A = 0;\ u_B = a_o \text{ and } v_B = b_o \qquad (9\text{-}5)$$

where the subscripts represent all the points in the indicated blocks.

(b) Homogeneous strain in the whole body and differential rotation. The deformation model is linear and it may be expressed directly in terms of the strain components (ε_x, ε_y, ε_{xy}) and differential rotation,ϖ, as:

$$u = \varepsilon_x x + \varepsilon_{xy} y - \varpi y$$

$$v = \varepsilon_{xy} x + \varepsilon_y y + \varpi x \qquad (9\text{-}6)$$

(c) A deformable body with one discontinuity, say, between blocks A and B, and with different linear deformations in each block plus a rigid body displacement of B with respect to A. Then the deformation model is written as:

$$u_A = \varepsilon_{xA} x + \varepsilon_{xyA} y - \varpi_A y$$

$$v_A = \varepsilon_{xyA} x + \varepsilon_{yA} y + \varpi_A x \qquad (9\text{-}7)$$

and

$$u_B = a_0 + \varepsilon_{xB}(x - x_0) + \varepsilon_{xyB}(y - y_0) - \varpi_B(y - y_0)$$

$$v_B = b_0 + \varepsilon_{xyB}(x - x_0) + \varepsilon_{yB}(y - y_0) + \varpi_B(x - x_0) \qquad (9\text{-}8)$$

where x_0, y_0 are the coordinates of any point in block B.

(4) Usually, the actual deformation model is a combination of the above simple models or, if more complicated, it is expressed by non linear displacement functions which require fitting of higher order polynomials or other suitable functions. If time-dependent deformation parameters are sought, then the above deformation models will contain time variables.

(5) A vector Δl of changes in any type of observations, for instance, changes in tilts, in distances, or in observed strain, can always be expressed in terms of the displacement function. For example, the relationship between a displacement function and a change ds in the distance observed between two points i and j in two monitoring campaigns may be written as:

$$ds_{ij} = [(x_j - x_i)/s]\ u_j + [(y_j - y_i)/s)]\ v_j -$$

$$[(x_j - x_i/s)]\ u_i - [(yj - yi)/s)]\ v_i \qquad (9\text{-}9)$$

where u_j, $v_{j,}$ $u_{i,}$ and v_i are components of the displacement function at points (x_j, y_j, and x_i,y_i), respectively. For a horizontal tiltmeter, the change dτ of tilt between two survey campaigns may be expressed in terms of the vertical component (w) of the displacement function as:

$$d\tau = (\partial w/\partial x)\ \sin\alpha + (\partial w/\partial y)\ \cos\alpha \qquad (9\text{-}10)$$

where α is the orientation angle of the tiltmeter.

(6) The functional relationships for any other types of observables and displacement functions are, in matrix form, written as:

$$\Delta\mathbf{l} = \mathbf{A}\mathbf{B}_{\Delta l}\ \mathbf{c} \qquad (9\text{-}11)$$

where **A** is the transformation matrix (design matrix) relating the observations to the displacements of points at which the observations are made, and $\mathbf{B}_{\Delta l}$ is constructed from the above matrix **B**(x, y, z, t-t_o) and related to the points included in the observables.

(7) If redundant observations are made, the elements of the vector **c** and their variances and covariances are determined through least-squares approximation, and their statistical significance can be calculated. One tries to find the simplest possible displacement function that would fit to the observations in the statistically best way.

(8) The search for the “best” deformation model (displacement function) is based on either a priori knowledge of the expected deformations (for instance from the finite element analysis) or a qualitative analysis of the deformation trend deduced from all the observations taken together. In the case of the observables being the relative displacements obtained from geodetic surveys, the

iterative weighted transformation of the displacements gives the best picture of the actual deformation trend helping in the spatial trend analysis. In the case of a long series of observations taken over a prolonged period of time, plotting of individual observables versus time helps to establish the deformation trend and the deformation model in the time domain. In the analysis, one has to separate the known deformation trend from the superimposed investigated deformation. For example, in order to distinguish between the cyclic (seasonal) thermal expansion of a structure with a one-year period of oscillation and a superimposed deformation caused by other effects which are, for instance, linear in time, all the measurements can be analyzed through a least-squares fitting of the cyclic function

$$y = a_1\cos(\omega t) + a_2\sin(\omega t) + a_3 t + a_4 + a_5\delta(t_i) + ..., \qquad (9\text{-}12)$$

to the observation data, where $\omega = 2\pi/yr$, and a_3 is the rate of change of the observation (extension, tilt, inclination, etc.). The amplitude and phase of the sinusoid can be derived from a_1 and a_2. The constant a_4 is the y-intercept and the constants a_5, ... are possible slips (discontinuities) in the data series and $\delta(t_i)$ is the Kronecker's symbol which is equal to 1 when $t \geq t_i$, with t_i being the time of the occurrence of the slip, and is equal to 0 when $t < t_i$.

(9) Summarizing, the geometrical deformation analysis using the UNB Generalized Method is done in four steps:

(a) The trend analysis in space and time domains and the selection of a few alternative deformation models which seem to match the trend and that make physical sense.

(b) The least-squares fitting of the model or models into the observation data and statistical testing of the models.

(c) The selection of the "best" model that has as few coefficients as possible with as high a significance as possible (preferably all the coefficients should be significant at probabilities greater than 95 percent) and which gives as small a quadratic form of the residuals as possible.

(d) A graphical presentation of the displacement field and the derived strain field.

(10) The results of the geometrical analysis serve as an input into the physical interpretation and into the development of prediction models as discussed above.

9-29. Statistical Modeling of the Load-Displacement Relationship

a. The statistical method establishes an empirical model of the load-displacement relationship through the regression analysis, which determines the correlations between observed deformations and observed loads (external and internal causes producing the deformation). Using this model, the forecasted deformation can be obtained from the measured causative quantities. A good agreement between the forecasts and the measurements then tells us that the deformable body behaves as in the past. Otherwise, as in the previous case, reasons should be found and the model should be refined.

b. Interpretation by the statistical method always requires a suitable amount of observations, both of causative quantities and of response effects. Let d(t) be the observed deformation of an object point at time t. For a concrete dam, for example, it can usually be decomposed into three components:

$$d(t) = d_H(t) + d_T(t) + d_r(t) \qquad (9\text{-}13)$$

where $d_H(t)$, $d_T(t)$, $d_r(t)$ are the hydrostatic pressure component, thermal component, and the irreversible component due to the nonelastic behavior of the dam, respectively. The component $d_H(t)$ is a function of water level in the reservoir and can be modeled by a simple polynomial:

$$d_H(t) = a_0 + a_1 H(t) + a_2 H(t)^2 + ... + a_m H(t)^m \qquad (9\text{-}14)$$

where H(t) is the elevation of the water in the reservoir. The component $d_T(t)$ can be modeled in various ways depending on the information on hand. If some key temperatures $T_i(t)$, (i = 1, 2, ..., k) in the dam are measured, then

$$d_T(t) = b_1 T_1(t) + b_2 T_2(t) + ... + b_k T_k(t) \qquad (9\text{-}15)$$

If air temperature is used, the response delay of concrete dams to the change in air temperature should be considered. If no temperature is measured, the thermal component can be modeled by a trigonometric function.

c. The irreversible component dr(t) may originate from a nonelastic phenomena like creep of concrete or creep of rock, etc. Its time-dependent behavior changes from object to object. It may be modeled, for example, with an exponential function. The following function is appropriate for concrete dams:

$$d_r(t) = c_1 t + c_2 \ln t \qquad (9\text{-}16)$$

The coefficients a_i, b_i, c_i in the above equations are determined using the least-squares regression analysis. The final model suggests the response behavior of the different causative factors and is used for prediction purposes.

d. For an earth dam, the thermal effect is immaterial and the irreversible component becomes dominant. It should be mentioned that the statistical method for physical interpretation is applicable not only to observed displacements, as discussed above, but also to other monitored quantities, such as stress, pore water pressure, tilt of the foundation, etc. The only difference is that the response function for each causative quantity may change.

9-30. Deterministic Modeling of the Load-Deformation Relationship

a. The deterministic method provides information on the expected deformation from the information on the acting forces (loads), properties of the materials, and physical laws governing the stress-strain relationship.

b. Deformation of an object will develop if an external force is applied to it. The external forces may be of two kinds: surface force, i.e., forces distributed over the surface of the body, and body forces, which are distributed over the volume of the body, such as gravitational forces and thermal stress. The relation between the acting forces and displacements d is discussed in many textbooks on mechanics. Let **d** be the displacement vector at a point and **f** be the acting force. They are related as

$$\mathbf{L}^T\mathbf{D}\mathbf{L}\mathbf{d} + \mathbf{f} = 0 \qquad (9\text{-}17)$$

where **D** is the constitutive matrix of the material whose elements are functions of the material properties (e.g., Young's modulus and Poisson's ratio) and **L** is a differential operator transforming displacement to strain. If initial strain ε_0 and initial stress σ_0 exist, the above equation becomes

$$\mathbf{L}^T\mathbf{D}\mathbf{L}\mathbf{d} + (\mathbf{L}^T\sigma_0 - \mathbf{L}^T\mathbf{D}\varepsilon_0) + \mathbf{f} = 0 \qquad (9\text{-}18)$$

In principle, when the boundary conditions, either in the form of displacements or in the form of acting forces, are given and the body forces are prescribed, the differential equation can be solved. However, direct solution may be difficult, and numerical methods such as the finite element or boundary element of finite differences methods are used. The finite element method (FEM) is the most commonly used method in structural and geotechnical engineering, particularly in modeling dam deformations.

c. The basic concept of the FEM is that the continuum of the body is replaced by an assemblage of small elements which are connected together only at the nodal points of the elements. Within each element a displacement function (shape function) is postulated and the principle of minimum potential is applied, i.e., the difference between the work done by acting forces and the deformation energy is minimized. Therefore, the differential operator **L** is approximated by a linear algebraic operator. Numerous FEM software packages are available in the market ranging significantly in prices depending on their sophistication and adaptability to various types of material behavior. One very powerful software package is FEMMA (Finite Element Method for Multidisciplinary Applications) developed at the UNB for 2-D and 3-D finite element elastic, visco-elastic, and heat transfer analyses of deformations. FEMMA has found many practical applications in dam deformation analyses, in tectonic plate movements, in ground subsidence studies, and in tunneling deformations.

d. In the deterministic modeling of dam deformations, the dam and its foundation are subdivided into a finite element mesh. The thermal component dT and hydrostatic pressure component dH are calculated separately. Assuming some discrete water level in the reservoir, the corresponding displacements of the points of interest are computed. A displacement function with respect to water level is obtained by least-squares fitting of a polynomial to the FEM-computed discrete displacements. Then, the displacements at any water level can be computed from the displacement function. In computation of the thermal components, the temperature distribution inside the structure should first be solved. Again, FEM could be used, based on some measured temperatures (boundary conditions). Both the coefficient of thermal diffusivity and the coefficient of expansion of concrete are required. The thermal components for the points of interest are calculated using FEM with computed temperature at each nodal point. The total deformation is the sum of these two components plus possible action of some other forces, e.g., swelling of concrete due to alkali aggregate reaction which can also be modeled with FEM.

e. FEM is, certainly, a powerful tool in the deterministic modeling of deformations. One has to remember, however, that the output from the FEM analysis is only as good as the quality of the input and as good as the experience of the operator who must have a good understanding of not only the computer operation but, particularly, good knowledge in the mechanics of the deformable bodies. One should not treat FEM as a magic (black box) tool.

9-31. Hybrid Method of Deformation Analysis

a. As one can see from the above sections, interpretation by statistical methods requires a large amount of observations, both of causative quantities and of response effects. Thus the method is not suitable at the early stage of dam operation when only short sets of observation data are available. In addition, some portions of the thermal and hydrostatic pressure effects may not be separated by the statistical modeling if the changes in temperature and in the elevation of water in the reservoir are strongly correlated. The deterministic method proves very advantageous in these aspects. The deterministic method is of an a priori (design) nature. It uses the information on geometric shape and material properties of the deformable body and acting loads to calculate deformations. However, due to many uncertainties in deterministic modeling such as an imperfect knowledge of the material properties, possibly wrong modeling of the behavior of the material (particularly when a nonelastic behavior takes place), and approximation in calculations, the computed displacements may depart significantly from the observed values d(t). In this case, if, for example, a suspicion is that the discrepancy is produced by uncertainties in Young's modulus of elasticity, E, and the thermal coefficient of expansion, α, the deterministic model can be enhanced by combining it with the statistical method, in the form

$$d(t) + v(t) = x d_H(t) + y d_T(t) + c_1 t + c_2 \ln t \qquad (9\text{-}18)$$

where v(t) is the residual, $d_H(t)$ and $d_T(t)$ are the hydrostatic and thermal components, respectively, calculated from the deterministic modeling, and the last two terms take care of the possible irreversible component. The functional model for the irreversible component may vary and can be changed by examining the residuals. The unknowns x, y, c_1, c_2 are estimated from the observations using the least-squares estimation. The coefficient x is a function of Young's modulus and y is a function of the thermal expansion coefficient of concrete:

$$x = E_0/E \qquad (9\text{-}19)$$

$$y = \alpha/\alpha_0 \qquad (9\text{-}20)$$

where E_0 and α_0 are the values used in the deterministic modeling.

b. There must be a calibration of the constants of the material properties using the discrepancies between the measured displacements of a point at different epochs and that calculated from FEM. One must be aware, however, that if the real discrepancy comes from other effects than the incorrect values of the constants (e.g., non-elastic behavior), the model may be significantly distorted.

c. Recently, as aforementioned, a concept of a global integration has been developed, where all three — the geometrical analysis of deformations and both methods of physical interpretation — are combined. Using this concept, deformation modeling and understanding of the deformation mechanism can be greatly enhanced.

9-32. Automated Data Management of Deformation Surveys

a. Advantages and limitations of automation. In the total effort of deformation monitoring, the quality of the analysis of the behavior of the object being monitored depends on the location, frequency, type, and reliability of the data gathered. The data concerned are any geotechnical observable as well as any conventional geodetic observable (angle, distance, or height difference). Apart from the location and type of instrumentation, the frequency and reliability of the data can be enhanced by employing an "automatic" system of data gathering or acquisition and processing (including the deformation analysis). A data management system encompasses everything that happens to the data from the instant at which it is sensed to the time of analysis. Under ordinary circumstances, the interval of time between sensing and analysis may extend over several days or more. Under critical conditions, this may have to be nearly instantaneous in order to provide a warning, if necessary. The volume of data may consist of only several items (in the simplest routine investigation) to many hundreds or thousands (in very complex, critical situations, particularly if vibration behavior is of interest). The rate of sampling may be annually, monthly, weekly, daily, hourly, or even more frequently. The amount of human involvement may range from total (a "manual" system) to virtually none (an "automatic" system). Neither extreme is practical. A manual system is labor intensive and liable to errors or blunders and is less flexible in the re-examination of data. An automatic system is attractive but has some limitations. Although a "data acquisition system" strictly involves the gathering of data, the phrase has been used by many to mean the whole system of data management. The weighed advantages and limitations of an automatic data acquisition system are summarized in the following two lists.

Advantages of an automatic data acquisition system are:

(1) Personnel costs for reading instruments and analyzing data are reduced.

(2) More frequent readings are possible.

(3) Retrieval of data from remote or inaccessible locations is possible.

(4) Instantaneous transmission of data over long distances is possible.

(5) Increased reading sensitivity and accuracy can be achieved.

(6) Increased flexibility in selecting required data can be provided.

(7) Measurement of rapid fluctuations, pulsations, and vibrations is possible.

(8) Recording errors are fewer and immediately recognizable.

(9) Data can be stored electronically in a format suitable for direct computer analysis.

The limitations of an automatic system are:

(1) A knowledgeable observer is replaced by hardware, i.e., less frequent "intelligent" visual inspections.

(2) An excess of data could be generated, leading to a failure in timely response.

(3) The data may be blindly accepted, possibly leading to a wrong conclusion.

(4) There could be a high initial cost and, possibly, a high maintenance cost.

(5) Often requires site-specific or custom components that may be initially unproven.

(6) Complexity may require an initial stage of debugging.

(7) Specialized personnel may be required for regular field checks and maintenance.

(8) A manual method is required as an alternative (backup).

(9) A reliable and continuous source of power is required.

(10) The system may be susceptible to damage by weather or construction activity.

With an appropriate compromise between manual and automatic functions, a properly designed and working system can readily minimize the effects of the limitations mentioned above. Therefore, the advantages of an automatic (really "semiautomatic") system easily outweigh its disadvantages.

b. The ENEL system. In the ENEL system (Italian Ente Nazionale per l'Energia Elettrica), there are two major subsystems: "Off-line" which serves as a central storage of all data and "On-line" in which most of the activity takes place. It is the On-line portion that is of interest and will be described here. Two minicomputers are involved and are linked for the teletransmission of data. The remote virtually duplicates the functions of the local, except for the actual capture of data. The local, "ESSDI/L," provides data acquisition, validation, processing, storage, and transmission to other sites. Also, it issues a warning if the observed effect differs from the expected effect (as derived from deterministic modeling) by more than an established tolerance.

c. The USBR system. This system is for the Calamus (Embankment) Dam managed by the U.S. Bureau of Reclamation. The main aspect of this system is the means of communication by telephone line or satellite link from the dam to various locations in the country.

d. The USACE system. This is a system used by the St. Louis District, with respect to automated acquisition, processing, and plotting of data. An example of typical dialogue encountered in using the minicomputer-based system is given. Another system is under development, initially to deal with totally automating piezometers in embankment dams, particularly with respect to local and district communication.

e. The UNB system. A data management system was devised by UNB for use on an IBM PC AT compatible on-site microcomputer. It was created to replace a manual system already in use for several years. In the field, a programmed data collector provides for direct connection to (and sometimes control of) instrumentation and for the keyboard entry for other equipment. The system can also accommodate manually recorded data or data directly acquired from instrumentation. The raw data are contained in observation files, archived for security, and are processed or "reduced" (using calibration, test values, etc.) into data files which are then used by various analysis and display software.

(1) In the field, there is a check file that is either accessed during data collecting or available in hard copy. The check file contains expected values predicted from stochastic (statistical) analyses of the data files and thus provides for a warning in the field. A warning is also given in the processing if the currently processed value differs from the most recent value in the data file, beyond a set tolerance.

(2) The major advantage of the UNB system is that any data or derived data, whether geotechnical or geodetic (so long as it has been repeated in a suitable time series), can be brought together in the integrated deformation analysis of a structure. In the process, a time series is analyzed for trend, with the separation of seasonal and long-term behavior. In the absence of actual temperature information, the trend ("y," the change in the value of an observed or derived quantity) against time ("t," in years) is described by the following equation

$$y = a_1 \sin\omega t + a_2 \cos\omega t + a_3 t + a_4 + a_5 + ...$$

in which

ω = 2π since a period of 1 year is assumed

a_3 = the "rate" or long-term trend

a_5 .. = possible values of slips accounting for discontinuities in the data series (a_4 is also a slip, but it is required so that the fitting is not unduly constrained)

and from which the amplitude and phase can be derived to provide a comparison of seasonal behavior among the various measurement points in the structure. Done rigorously using the method of least squares, the fitting provides a full statistical analysis of the trend with the detection of outlying or erroneous data. It is possible to derive a new series from two original series or to create a series from repeated geodetic campaigns (e.g., tilt derived from leveling). The system can also show several series of data simultaneously, without fitting, to provide a graphical comparison of the series.

(3) The treatment of the data can be described as follows: The geodetic data are treated traditionally in campaigns for adjustment ("C.A.") and spatial trend analysis ("S.T.A."). Once the observations have been repeated a sufficient number of times, they can be treated as a time series ("T.S.A" time series analysis and plot, e.g., tilt from leveling; "S.S.A." spatial series analysis and plot, subsidence). Geotechnical series are treated in a similar manner ("T.S.P." time series analysis and plot, "S.S.P." spatial series analysis and plot, e.g., borehole profile changes). The time series analyses ("T.S.A." and "T.S.P.") are fitted with the sinusoid in the above equation to separate seasonal effects from the long-term trend. All of the trend analyses are automated by command files which are set up to control fitting and automated plotting of several series in succession. All of the data can be used together in simultaneous integrated geometrical analyses following the UNB Generalized Method (e.g., "D.M."). If desired, several series can be plotted simultaneously, without fitting. Since both the observation files and the data files are ASCII text files, they are accessible also through any text editor for manual entry or editing and can be input to other applications.

f. Desirable characteristics of an automated system. Overall, the desirable characteristics of a data management system for deformation surveys (including both geotechnical and geodetic observables) include:

(1) Data integrity (offering checks in the field and later processing).

(2) Data security (automatic archiving and regular data file backup).

(3) Automation of acquisition, processing, and analysis.

(4) Compatibility and integration with other observables.

(5) Flexibility in access to the data for possible manual entry and editing.

(6) Data openness (useable by other software).

(7) Flexibility in the system to be easily modified to accommodate additional instrumentation or other forms of analysis.

(8) On-site immediate access to data or any of the forms of analysis.

(9) Near-real time results of trend or other analyses.

(10) Testing and calibration is an integral component of the system.

9-33. Standards and Specifications for Dam Monitoring

a. Although there are significant differences between various countries in the quality of monitoring deformation, there is no one country which can serve as an example for others concerning all the three main aspects of dam deformation monitoring, i.e. monitoring techniques, design of monitoring schemes, and analysis and management of the collected observations.

b. A few countries, mainly eastern European countries, during the time when they were still part of the "communist block," including China, developed some national standards and specifications for dam deformations. Unfortunately, these specifications were developed for practically unrealistic conditions under government dictatorship and ownership of all dams in their countries. Although some of the standards and specifications may be technically acceptable, they would have to be carefully reviewed and extensively modified for practical use. Although there are some reputable books on specialized geodetic instrumentation, there is no up-to-date manual or book which discusses all of the aspects of geodetic monitoring surveys, particularly the design and processing of the geodetic monitoring networks.

c. Although it seems to be unrealistic and practically impossible to prepare overall detailed standards and specifications for dam monitoring at the national level, certain processes could, and perhaps should, be standardized, particularly:

(1) Calibration of instruments.

(2) Procedures for the integrated geometrical analysis and data management from the moment of the data collection, through the reduction of data and trend analysis (including the identification of the unstable reference points) to the determination of the deformation model.

d. General guidelines are required with respect to: design of the integrated monitoring schemes, particularly optimal choice of instrumentation, selection of the parameters to be monitored, the required accuracy, optimal location of the instruments, and frequency of measurements.

9-34. Monitoring Techniques and Their Applications

a. With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any, practically needed, instrumental resolution and precision, full automation, and virtually real-time data processing. Remotely controlled telemetric data acquisition systems, working continuously for several months without recharging the batteries in temperatures down to -40˚C are available at a reasonable cost. Thus, the array of different types of instruments available for deformation studies has significantly broadened within the last few years. This creates a new challenge for the designers of the monitoring surveys: what instruments to choose, where to locate them, and how to combine them into one integrated monitoring scheme in which the geodetic and geotechnical/structural measurements would optimally complement each other.

b. As far as the actual accuracy of deformation surveys is concerned, the main limiting factors are not the instrument precision but the environmental influences and negligence by the users, namely:

- Influence of atmospheric refraction (all optical and electro-optical measuring systems).
- Thermal influences, affecting the mechanical, electronic, and optical components of the instruments (in any type of instrumentation) as well as the stability of survey stations.
- Local instability of the observation stations (improper monumentation of survey stations and improper installation of the in situ instrumentation).
- Lack of or improper calibration of the instruments.

- Lack of understanding by the users of the sources of errors and of the proper reduction of the collected observations.

Most of the above-listed effects could be eliminated or drastically reduced if the monitoring surveys were in the hands of qualified professionals.

c. The problem of calibration is very often underestimated in practice, not only by the users but also by the manufacturers. For long-term measurements, the instrument repeatability (precision) may be affected by aging of the electronic and mechanical components resulting in a drift of the instrument readout. Of particular concern are geotechnical instruments for which the users, in general, do not have sufficient facilities and adequate knowledge for their calibration. The permanently installed instruments are very often left in situ for several years without checking the quality of their performance.

d. The last aspect, the lack of understanding of the sources of errors affecting various types of measurements and the proper data handling is, perhaps, the most dangerous and, unfortunately, the most frequent case in measurements of deformations. The measurements and processing of the monitoring data, particularly geodetic surveys, are usually in hands of technicians who may be experienced in the data collection, but have no educational background in handling and reducing the influence of various sources of errors. In this case, even the most technologically advanced instrumentation system will not supply the expected information.

e. A worldwide review of the monitoring techniques used in monitoring surveys indicates that, generally, there are no significant differences between different countries in the employed instrumentation. There are, however, differences in the accuracy requirements, the required frequency of observations, and in the details concerning the use of the instruments. This is also true among USACE districts. Strong biases toward either geodetic or geotechnical instrumentation have been developed in individual countries. The biases are correlated with the level of educational background in geodetic surveys. For instance, in Switzerland and Germany, where the standard of geodetic surveying education and the number of specialists in surveying engineering are high, the geodetic surveying techniques play the dominant role in monitoring large dams. Whereas, in the countries like Italy, France, and U.K., where the education in surveying engineering does not have a long-standing tradition, the geotechnical techniques are mostly used. ICOLD seems to be biased toward use of geotechnical/structural instrumentation rather than geodetic. This is dictated, perhaps, by the fact that the members of the ICOLD Committee on Monitoring Dams and Their Foundations are predominantly specialists in geotechnical instrumentation with some obsolete views on the use geodetic techniques. With the exception of a few individual dams, there is no country which takes full advantage of the optimal combination of both techniques, i.e., the concept of the integrated monitoring surveys developed within the activity of FIG. The biases toward one or another type of techniques are obviously produced by a lack of specialists with full knowledge of both geotechnical and geodetic measurements. This leads to the following:

(1) The concept of the integrated monitoring systems, in which the geotechnical and geodetic surveying techniques complement each other, should be made known to all the owners of large dams through the aforementioned efforts of the national committees on large dams and publication of guidelines.

(2) Monitoring schemes for all new dams should be designed at the design stage of the dam.

(3) As far as the new monitoring technologies are concerned, more research is still needed in:

(a) Optimal use of GPS.

(b) Automation of data acquisition including the optimal selection of electronic and optical transducers, comparison of their performance (sources of errors, durability), development of calibration methods, and

(c) Application of new technologies, for example the use of the optic-fibre sensors, CCD sensors, etc.

9-35. Analysis and Modeling of Deformations and Their Applications

a. Over the past 10 years there has been a significant progress in the development of new methods for the geometrical and physical analyses of deformation surveys. FIG has been leading in the developments, particularly in the areas of integrated geometrical analysis of structural deformations and combined integrated analysis. However, due to the aforementioned lack of the interdisciplinary cooperation and insufficient exchange of information, FIG developments have not yet been widely adapted in practice. Therefore, the general worldwide use of the geometrical analysis methods is still poor, including even the basic analysis of geodetic monitoring networks. The latter is of a particular concern in the United States where there is a shortage of qualified surveying engineers. The worldwide situation with the physical analysis is much better with most countries who utilize both deterministic and statistical methods for modeling and interpreting deformations at various levels of sophistication. However, most countries do not take full advantage of the developments in the integration of the observed deformations with deterministic models to enhance the latter ones. Also only few countries utilize the observed deformations to develop prediction models through the regression analysis. Italy seems to lead in the use of the combined statistical and deterministic modeling. Canada, within the activities of FIG, leads in the development of new concepts in the global integrated analysis. For example, the UNB has developed new methodology and software for the integrated geometrical analysis, and for the finite element deterministic modeling and prediction of deformations.

b. The above comments lead to the following:

(1) The analysis of deformation surveys should be in hands of interdisciplinary teams consisting of geotechnical, structural, and surveying engineers specialized in both geometrical and physical analyses.

(2) More use should be made of the concepts and developed methodologies for the geometrical integrated analysis and combined deterministic/statistical modeling of deformations.

Chapter 10
Standards and Specifications for Deformation Monitoring Reference Networks

10-1. General Scope

This chapter details USACE standards and specifications for establishing external deformation monitoring reference networks, from which periodic PICES observations are made to structure object/target points. Also included are requirements and procedures for periodically monitoring the stability of the reference network points. The standards and procedures for establishing reference networks are intended to support the Reference Line Ratio (RLR) technique, also known as the "Robertson Method," of periodic structural monitoring. The RLR/Robertson technique is recommended for most USACE PICES monitoring applications and is further described in Chapter 11.

a. Accuracy standards and specifications. The accuracy standards developed in this chapter apply to deformation surveys carried out by conventional, photogrammetric, and GPS survey methods. It is possible that two or even three of these methods can be used together; therefore, the standards are based upon the magnitude of a detectable single point movement and are independent of the method used. Alternately, the specifications sections are treated as a function of the procedures required, rather than by method used. Both the standards and specifications developed are adapted from a consolidation of information from various USACE commands and other deformation monitoring agencies.

b. Geodetic survey methods. The standards and specifications are based on the determination of deformations using traditional geodetic survey techniques and equipment, as observed from reference networks. The traditional geodetic survey accuracy "orders" (i.e., First-Order, Second-Order) do not apply to deformation surveys since most horizontal and vertical observations are of relatively short length (less than 1,000 m). Generally, traditional geodetic First-Order techniques (not accuracy standards) are followed in PICES surveys.

10-2. Reference Network Accuracy Standards

Table 10-1 depicts the accuracy required in a reference network to obtain the target accuracies outlined in Chapter 9.

Table 10-1
Reference Network Relative Accuracy for Concrete and Earthen Embankment Structures

Linear Extent of Network	Concrete (mm)	Embankment (mm)
Absolute External <u>Observations</u>		
100-1,000 m	5	15
1 km - 5 km	20	50
Relative Deflection Observations <u>(Micrometers)</u>		
10 m	0.5	2
100 m	1	13

Linear extent of deformation network is the maximum spatial distance between the two most widely separated points in the network.

a. From Table 10-1, the relative accuracies of reference points spaced at a typical distance of 1 km are 5 mm for a concrete structure and 15 mm for an embankment structure. These relative accuracies can be readily achieved using careful geodetic survey techniques and instrumentation. This includes use of traditional theodolites and EDM, or full use of modern electronic total stations. Centering errors are minimized through use of forced-centering instrument/target mounts. Use of the RLR/Robertson method minimizes refractive errors in EDM measurement.

b. The relative accuracies of alignment reference points used for short-range deflection observations (usually with micrometers) are as indicated. The use of forced-centering is critical for obtaining repeatability in deflections, and the absolute relative accuracy indicated is only representative should these points be used for external monitoring.

10-3. Accuracy Requirements for Surveying PICES Targets

Table 10-2 depicts the recommended accuracy by which target points on the structure must be located during periodic PICES surveys taken from the reference network. Accuracies indicated are absolute -- relative to the entire reference network, not over a single line (which will be far more precise).

a. Concrete structures. To obtain the indicated 10-mm accuracy, traditional geodetic survey procedures

Table 10-2
Typical Accuracy Requirements for PICES Surveys

Concrete Structures	
Dams, Outlet Works, Locks, Intake Structures	
Long-Term Movement (Geodetic survey methods)	10 mm
Relative Short-Term Deflections Crack/Joint Movements Monolith Alignment (Precision micrometer alignments)	0.01 in. (0.2 mm)
Vertical Stability/Settlement (Precise geodetic leveling)	2 mm
Embankment Structures	
Earth-Rockfill Dams, Levees	
Slope/crest Stability (Total station/DGPS)	0.1 foot
Crest Alignment (Total station/DGPS)	0.1 foot
Settlement measurements (Differential leveling)	0.05 foot
Control Structures	
Spillways, Stilling Basins, Approach/Outlet Channels, Reservoirs	
Scour/Erosion/Silting (Hydrographic surveys)	0.2 to 0.5 foot

are employed. Use of the RLR/Robertson method will easily meet this accuracy standard. Relative accuracies over individual lines will be far more precise. Precise EDM and/or electronic total stations with forced-centering mounts are recommended for performing periodic observations.

b. Embankment structures. Most conventional precise survey methods will provide the accuracies required for these structures. Force-centered electronic total stations are recommended, although tripod-mounted theodolites/EDM will provide adequate results.

10-4. Local Coordinate Systems for Reference Network Surveys

The specifications outlined in this chapter refer to a 3-D local coordinate system where x and y are the horizontal coordinates and *z* is the vertical (i.e., height) coordinate. The X-Y grid system used is usually a local coordinate system or construction system. Connections to geographic systems or SPCS are not required or needed for PICES deformation surveys. The *z* coordinate can either be: local orthometric height if conventional survey or conventional combined with photogrammetric survey observations are used; local Euclidean height if photogrammetric survey observations are used only; or the local ellipsoidal height if only GPS, GPS plus conventional, and/or photogrammetric survey observations are used.

10-5. First-Order 3-D Deformation Monitoring Specifications

The following sections detail the minimum specifications that should be followed to achieve the reference network accuracies detailed in Table 10-1.

10-6. Network Design

When designing a network for deformation monitoring, it is important to keep in mind that: a structure and its foundations form a whole that is embedded in the surrounding terrain which may also impact the observations; abnormal structural behavior may occur either quite rapidly or gradually over time; and if abnormal deformation does occur, analysis of deformation measurement data can help to identify its cause. Therefore, a network design should be capable of monitoring both short- and long-term influences, as well as be capable of distinguishing between movement of the structure, its foundation, and surrounding terrain. Analysis of behavior for the short term consists of collection and analysis of specific data frequently, while long-term behavioral analysis consists of collecting and examining more differentiated data. Instrumentation and procedures for short-term analysis must be easy to operate and follow, providing measurements that are relatively easy to interpret. Instrumentation and procedures for long-term analysis are conducted less often, therefore instrumentation and procedures are typically more sophisticated, often requiring the aid of specialists in order to be done properly. The specific design of the deformation network can be used to determine the network configuration, instrumentation, and accompanying procedures to be followed.

a. There are no existing PC-based software packages specifically intended for deformation network design. Even though this is the case, network adjustment software using rigorous adjustment methods (e.g., method-of-least-squares-based adjustment software) can be used to carry out deformation network design through preanalysis of alternative designs. Deformation network design is then facilitated through a “sensitivity analysis” in which prechosen movements are tested to determine if they are detectable with the particular design alternative. Using network adjustment software in such a manner should be left to those with experience in such and should not be done by the novice.

b. Recognizing the complexity and difficulties of using network adjustment software for deformation network design, the following empirical relationship was developed and will be used to determine the detectable single point movement possible in a particular deformation network design:

$$S_p = P_p * \sqrt{2} \tag{10-1}$$

where

S_p = detectable single point movement

P_p = precision of the point position

P_p can be further defined based on the definition of the points position. For a 1-D (i.e., z) point position definition:

$$P_p = S_z \tag{10-2}$$

For a 2-D (i.e., x and y) point position definition:

$$P_p = \sqrt{(S_x^2 + S_x^2)} \tag{10-3}$$

For a 3-D (i.e., x, y, and z) point position definition:

$$P_p = \sqrt{(S_x^2 + S_y^2 + S_z^2)} \tag{10-4}$$

where

(for Equations 10-1, -2, and -3):

S_x, S_y, S_z = estimated precision, at the 95 percent confidence level of the respective x, y, and z coordinates

c. In lieu of network adjustment software that use a rigorous method of adjustment, Equations 10-1 through 10-4 will be used as the basis for deformation network design. The explanation for the use of a detectable single point movement for deformation network design should be obvious: a network is less sensitive to the movement of one point (i.e., it is more sensitive to the movement of a group of points); therefore, the magnitude of a single point is used to define the criteria for the deformation network design for deformation surveys.

d. In a network observed by conventional survey methods, there can be weak areas with respect to the precision, reliability, and detectability of movements because of the lack of redundant measurements. Subsequently, the network design for a network observed by conventional survey methods is very critical. By contrast, a network observed by photogrammetry, GPS, or a combination thereof typically provides a large number of redundant measurements and its network design is not as critical. Regardless of this phenomenon, the empirical relationships in Equations 10-1 through 10-4 are valid for deformation network design, exclusive of the method used to monitor it, and will be used when practicable.

10-7. Monumentation

A monument used for deformation monitoring is any structure or device which serves to define a point in the deformation survey network. It must have long-term stability with respect to the area surrounding it of less than 0.5 mm both horizontally and vertically. A monument can be classified as either a reference point or an object point. A reference point typically is not located on the deformation structure and is to be “occupied” during the deformation survey. An object point is located on the deforming structure and is to be “monitored” during the deformation survey. The monumentation described in the following paragraphs will be used for both horizontal and vertical observations as these observations are taken during a typical deformation survey. Therefore, the use of deeply set monuments used for either horizontal or vertical control surveys alone typically are not used in a deformation network design and subsequent observations.

10-8. Reference Point Monuments

a. Reference points installed in the earth shall be installed so as to have a depth equal to at least twice the depth of frost penetration in the project area. These reference points can be either a steel pipe pile or cast-in-place reinforced concrete pile (Figure 10-1 a and b). If a steel pipe pile is used, the nominal diameter will be no less than 20 cm, while the wall thickness will be no less than that for standard weight pipe. If a cast-in-place reinforced concrete pile is used, the nominal diameter will also be no less than 20 cm.

b. Installation of the reference point will be as follows:

(1) Steel pipe pile. A steel pipe pile will be installed by driving it until refusal. If refusal occurs at a depth of less than twice the depth of frost penetration in the project area, the pile will be removed and its

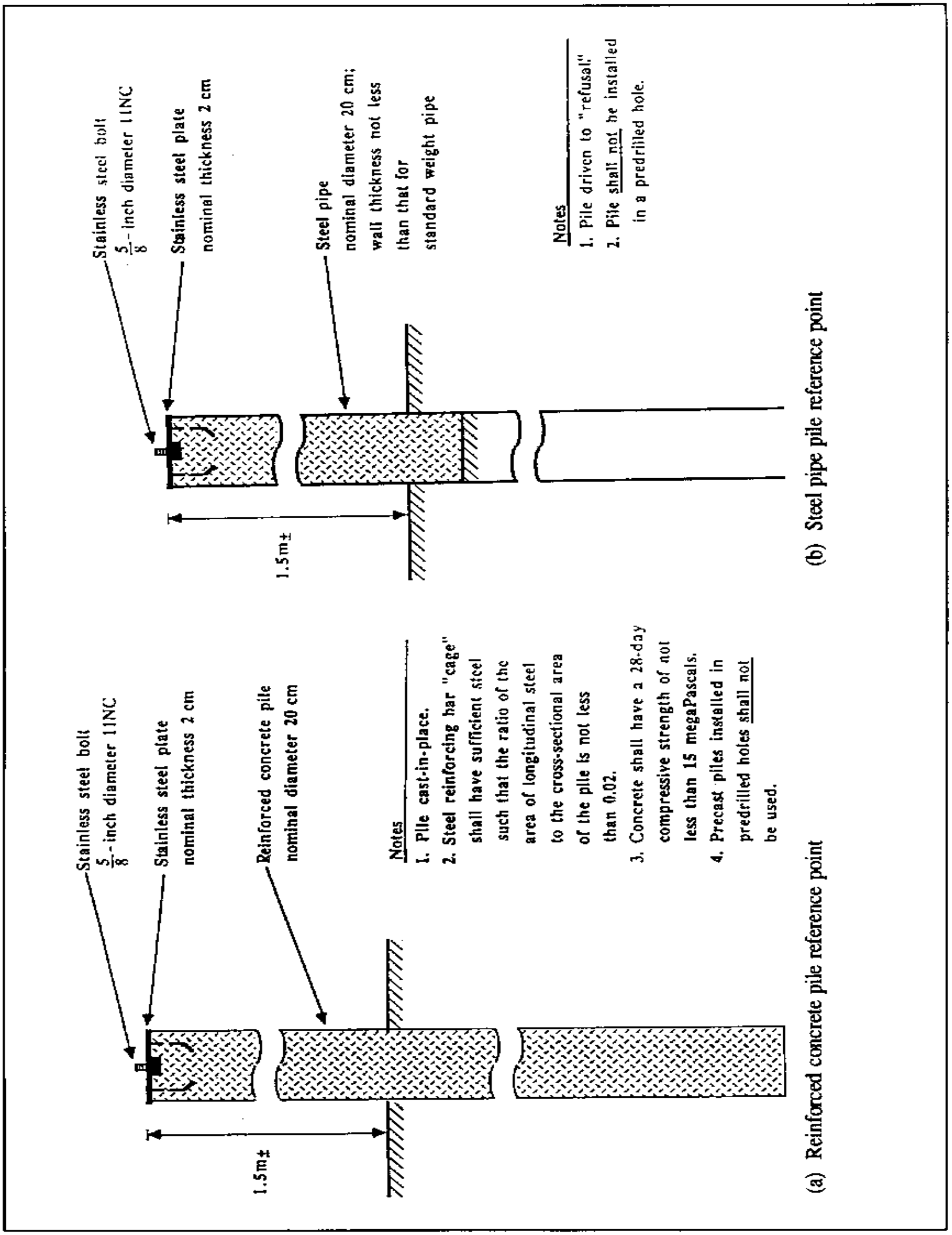


Figure 10-1. Reference points

installation attempted in another location. Steel pipes placed in oversized predrilled holes and backfilled will not be used as reference points.

(2) Cast-in-place reinforced concrete pile. A cast-in-place reinforced concrete pile will be installed by first drilling a hole to at least twice the depth of frost penetration in the project area. The cage of steel reinforcing bars used will have a cross-sectional area of steel to concrete of not less than 0.02. After the cage is formed, it is placed in the hole. Concrete with a 28-day compressive strength of not less than 15 megaPascals is then poured into the form. Precast reinforced concrete piles driven into predrilled holes or placed in oversized predrilled holes and backfilled will not be used for reference points.

c. If the length of line of observation is less 1 km, it is preferable for the reference point to extend above ground level to a convenient height (e.g., 1.5 m) where the equipment can be force centered. Typically, at the top of such a reference point pile, a stainless steel plate not less than 2 cm thick is cast into the top of the pile using a minimum of four steel reinforcing bar anchors welded to the underside of the plate. In the center of the plate, a 5/8-inch-diameter 11NC steel bolt is welded to the plate to allow for survey equipment to be attached.

d. Where cold weather conditions dictate, an insulation sleeve may need to be installed around the reference point pile that extends above the ground. The installation of a sleeve is to eliminate the possibility of temperature-induced pile movements that may be the result of solar radiation (i.e., temperature variation due to time of day). When this is the case, the sleeve should have an R value of not less than 10.

e. For pipe piles terminating at or slightly below ground level, a convex stainless steel plate and stub will be installed as described above. The plate will be convex as required for leveling observations and will have an etched cross at the highest point of the convex surface for horizontal observation. It is recommended that such piles also have a cylindrical rim and cover around it for protection. If a cylindrical rim and cover is used, it is further recommended the cover be buried so as to be easily recoverable with a metal detector, as well as to minimize the chance of vandalism.

f. If possible, the reference points should be installed at least a year prior to their use to minimize the effects of pile rebound and shrinkage. If this is not practicable, no less than a month prior to their use will suffice.

g. Reference points installed in rock or concrete shall consist of a stainless steel plate as described above, except with a steel reenforcing bar stub welded to the underside. For installation, a hole at least 50 percent larger than the stub will be drilled into sound rock or concrete. The plate with the stub attached will be secured to the rock or concrete using adequate epoxy adhesive to completely fill the void between the stub and the rock or concrete.

10-9. Object/Target Point Marks

a. Object points installed in the earth will consist of a nominal 3-m length of square steel hollow structural section with a nominal side length of 5 cm and a wall thickness not less than that for a standard weight square steel hollow structural section (Figure 10-2). The base of the section is sharpened by cutting it at a 45-degree angle. Welded approximately 15 cm from the base is one length of 10-mm-thick 20-cm-diameter circular helix with a pitch of 7 cm. Welded to the top of the pipe is a steel plate not less than 5 mm thick. In the center of the plate a 5/8-inch-11NC steel bolt onto which survey equipment is to be connected is drilled through and welded to the plate. If practicable, some method (e.g., through the use of a cap) should be used to protect the threads of the bolt during the time survey equipment is not attached.

b. Object points set directly in rock or concrete may be either a steel bolt onto or a steel insert into which survey equipment is force centered. The installation of these types of object points will be as follows:

(1) Steel bolt. The steel bolt will be drilled through and welded to a 5-cm-diameter, 1-cm-thick steel plate. A steel reinforcing bar stub of suitable length will be welded to the head of the bolt. A hole approximately 50 percent larger than the stub will be drilled in sound rock or concrete. The plate with the stub attached will be secured to the rock or concrete using adequate epoxy adhesive to completely fill the void between the stub and the rock or concrete. Once again, if practicable, some method (e.g., through the use of a cap) should be used to protect the threads of the bolt during the time survey equipment is not attached.

(2) Steel insert. Steel inserts have been designed by various manufacturers as off-the-shelf items. Manufacturer instructions for proper installation of the insert should be followed.

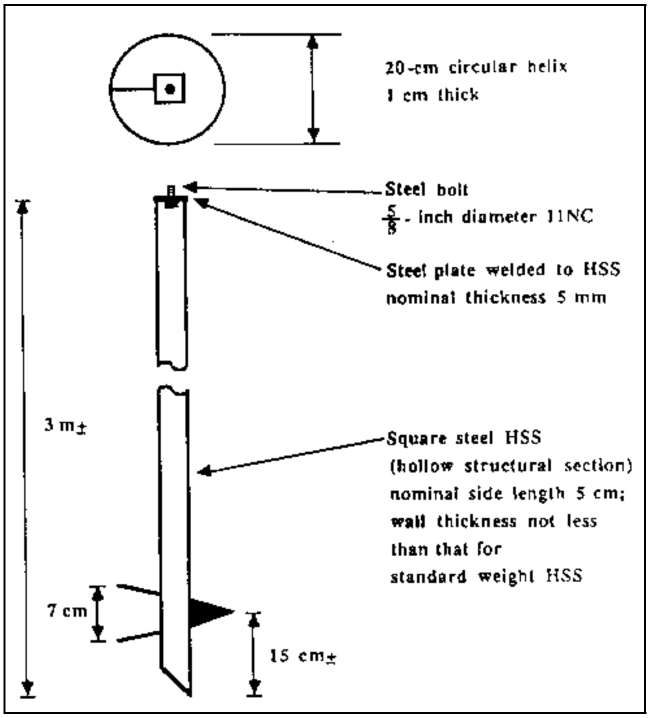


Figure 10-2. Hollow steel structure with helix base object point

c. Object points on materials (e.g., steel, masonry, etc.) other than described in the previous paragraphs will be permanently affixed. For object points to be mounted on steel, a steel bolt welded to the steel may be suitable. For masonry, or other material, a steel bolt, plate, and rear stub or a steel insert may be suitable.

d. For all reference and object points installed, an applicable identifier (e.g., numeric or alphanumeric) will be stamped on the point as appropriate. A permanent record will be kept of the identifier, description, location, and condition of each reference and object point.

e. Further information on specific monument design and installation is provided in EM 1110-1-1002.

10-10. PICES Targets

A target used for deformation monitoring is a device with a well-defined aiming point which is placed vertically over or attached to a monument. The purpose of a target is to permit making measurements to the point over which it is installed when the deformation survey network is observed. Such a device typically is installed only for the period of the survey. In some cases, the monument may be a target itself.

a. Targets used for angular measurement, either horizontal, vertical, or both, will be either:

- Standard force-centered target sets designed for one-second theodolites;
- Standard force-centered target set/prism combination used with a particular total station or theodolite and EDM;
- A target made of the material from which typical force-centered prisms are made;
- The monuments themselves.

b. Target set/prism combinations not matched to a particular total station will not be used. Also, target set/prism combinations for total stations which are noncoaxial will be tilting target set/prism combinations so the target set/prism can be tilted together to allow for alignment with the line of observation.

c. Targets for taped distances will be the monuments themselves.

d. Targets for EDMs will be the prisms included with the EDM. Prisms not matched to a particular EDM will not be used.

e. Targets for spirit-leveled height difference measurements will be the monuments themselves. If the monuments are steel inserts, the targets will be stainless steel plugs designed for the purpose. If more than one plug is to be used on a project, the plugs will be of the same size.

f. The preferred targets for photogrammetric-based deformation surveys typically consist of a white dot on a black background. The diameter of the white dot is chosen so as to yield an average image diameter of 60 microns. The black background typically is five times the diameter of the white target.

g. The targets for GPS-based deformation surveys generally are the monuments themselves.

10-11. PICES Instrumentation

The instruments typically used for deformation surveys are described in the following paragraphs.

a. If using an optical theodolite, it shall have a telescope magnification of 30 times or better, a plate level with a sensitivity of 20 seconds per 2-mm graduation or better, an automatic vertical circle compensator, and a coincident micrometer direct to 1 second or better.

b. If using an electronic theodolite, it will have the same characteristics of an optical theodolite or better, but the circle reading system will be accurate to 3 seconds or less.

c. Distances of 10 m or less can be measured with a steel or invar tape. An EDM will not be used to measure distances less than 10 m. Distances of 30 m or less can be measured with a tensioned steel tape, invar tape, or invar wire which can be attached to the steel bolt or insert directly, a subtense bar, or an EDM. An EDM is the preferred instrument for distances beyond 30 m. Microwave-based EDM systems will not be used. It is not necessary to measure any distances with multiple wavelength-based instruments.

d. Barometers will be capable of 2-mm mercury precision or better. Thermometers will be of 1 degree Celsius readings or better.

e. If spirit leveling is done, the instrument will be an automatic level with telescope magnification of 40 times or better, compensator with a sensitivity of 10-second per 2-mm level vial graduation, parallel plate micrometer capable of 0.1-mm readings. The compensator will be a free suspension one and not a mechanical one so as to minimize the effect of possible electromagnetic fields. The rod to be used should be an invar, double scale rod or one with a permanently attached circular level attached - both having graduations equal to the range of the parallel plate micrometer.

f. All equipment will have an optical plummet either incorporated in it or be capable of being used with a detachable tribrach that has an optical plummet.

g. When performing photogrammetric-based deformation surveys, only metric cameras will be used. Typically, only one camera is necessary as it is moved from station to station. The instrument used for image coordinate measurement (e.g., monocomparator, stereocomparator, or analytical stereocomparator) will be capable of 1-micron or better resolution.

h. When performing GPS-based deformation surveys, the receiver used will be at least geodetic quality, multichannel, single frequency, and capable of at least one-minute data sampling. At the least, the receiver should be capable of recording the GPS carrier frequency, receiver clock time, and signal strength for each data sample. A receiver is required for each reference and object point. The same receiver/antenna combination should be used for each setup.

10-12. Equipment Adjustment and Calibration

All equipment used for deformation monitoring surveys will be maintained in adjustment and calibration between use so as to minimize possible errors that may result from the equipment being out of adjustment. Manufacturer specifications will be used as the basis for the adjustments and calibrations.

a. No adjustments need be made to barometers, thermometers, and EDMs. Instead, systematic instrument errors will be determined by calibration and standardization procedures and the associated observations made with the equipment will be corrected accordingly.

b. If a tribrach is used, the only adjustment and calibration necessary is to the optical plummet if so equipped.

c. A metric camera will not be adjusted or calibrated during a survey session. If adjustment or calibration need be done, it will be done only by personnel qualified in such. The same applies for the image-coordinate measurement devices. Such activities should be done as a matter of habit at least once a year or more if necessary.

d. GPS equipment typically does not require adjustment and calibration. If a piece of GPS equipment does not appear to be operating correctly, the manufacturer should be consulted.

10-13. Survey Procedures

Formalizing a particular procedure to be followed that meets all circumstances is difficult, but generalized procedures can be developed and are detailed in the following paragraphs.

a. Angle measurement with a theodolite. When using a theodolite for angle measurement, it will be accurately plumbed over the occupied point by either attaching the theodolite to the point with a tribrach or using a tripod and tribrach with an optical plummet, as applicable. Once measurements are made, the level of the instrument will be checked. If found to be greater than 10 seconds, the measurements will be done again with a leveled instrument. When an electronic theodolite is used, the level of the instrument will be kept to less than 2 minutes.

(1) The reticule should be focused first and then the objective lens of the theodolite.

(2) If possible, observation should be limited to days when the weather conditions are fairly neutral (e.g., cloudy day with a light breeze). Days with temperature extremes should be avoided. If an instrument must be used when the temperature is hot, it should be protected from the sun by an umbrella.

(3) Observation of zenith angles, where necessary, should be limited to between 10 in the morning and 3 in the afternoon in order to minimize the chance for vertical refraction. When elevation differences are determined by zenith angles with a theodolite, such equipment will not be operated near high electromagnetic producing devices (e.g., transformers, high voltage power lines, etc.).

(4) Directions and zenith angles will be observed in four rounds. In all theodolites, face left and face right point and reads will be made on all targets. The exception is a few types of theodolites that have auto calibration based on point and reads to one target. All horizontal and vertical circle readings will be recorded manually or electronically to 0.1 second.

b. Distance measurement with a tape. Distances measured between monuments will be made point to point whenever possible. When using a steel or wire invar tape, the uncorrected distance to the millimeter, temperature, tension applied, slope, unsupported length(s), and the standardization error of the tape will be recorded. If unable to measure point to point, a tripod and theodolite will be plumbed and leveled over the points and the distance measured between the trunnion axis of the setup. If tensioned equipment is used, in addition to the previous measurements to be made, the uncorrected distance should be measured to 0.01 mm. Distance measurements made by tape will be independently made at least two times by repeating the setup required to make the measurements.

c. Distance measurement with a subtense bar. If measuring the distance with a subtense bar, the subtense bar and theodolite will be plumbed and leveled over the points defining each end of the line of observation as described in the previous paragraphs. The optical sight will be used to set the subtense bar perpendicular to the line of observation. The angle subtended by the subtense bar will be measured reiteratively four times by the theodolite. Do not forget to record the height of the instrument (i.e., HI) and height of the target (i.e., HT) to at least 0.5 cm for later reduction of the distance to point to point.

d. Distance measurement with an EDM. If measuring the distance with an EDM (including those incorporated within total stations), the EDM will be accurately plumbed and leveled over the point, or force-centered. Observations with an EDM will not be conducted over terrain where extremes may be present (e.g., across a valley or river) that under adverse weather conditions may produce large errors related to atmospheric conditions and temperature along the lines of observations. To this end, observations with an EDM also should be limited to days when favorable atmospheric conditions (e.g., slightly cloudy with a light breeze) are prevalent. An EDM should never be operated near external electromagnetic field producing sources.

(1) Prior to its use, an EDM should be allowed to “warm up” according to manufacturer specifications. Also, the EDM should be operated in the manufacturer recommended range of ambient temperatures (e.g., typically -20 and +40 °C) and with fully charged batteries.

(2) Prior to measurements with the EDM, the target prism will be set perpendicular to within 10 degrees of the line of observation. When actual measurements are made, the prism will be adjusted to maximize the strength of the signal.

(3) Each EDM measurement between object and target station will be done iteratively at least four times by resetting the instrument and performing the observation again.

e. Pressure and temperature measurement. Pressure and temperature will always be measured at the instrument stations and at the target station if it is a reference network point (rather than an object point). (Note that the RLR/Robertson Method is independent of pressure/temperature measurements.) Temperature and pressure will always be measured in a location shaded from the sun, exposed to any wind, and at least 5 feet above the ground and away from the observer and instrument. Corrections to temperature and pressure will never be applied when the observations are made, but will be applied when the observations are reduced.

f. Additional observation requirements. When observations are made, in addition to the other standard information required for a survey, the following should be recorded in the field book:

(1) Pressure to 2 mm.

(2) Temperature to nearest 1 °C.

(3) HI of ED and prism to 0.5 cm, HI of theodolite and target to 0.5 cm if distance is measured, 0.5 mm if both distance and height differences are measured.

g. Leveling operations. When determining elevation by precise spirit leveling, the following guidelines will be followed:

(1) Either one or two double-scale invar rods will be used. For short runs, traditional three-wire procedures are allowable.

(2) Level lines will be run in only one direction.

(3) Either observed stadia or a cloth tape may be used to balance the foresight and backsight distances between two deformation survey network points. If the distances cannot be balanced, they will be recorded so that the height difference can be adjusted during data reduction.

(4) If using one level rod, it will be moved from backsight to foresight as quickly as possible and readings made. The readings will be recorded manually in the field book or electronically to 0.01 mm.

(5) The maximum length of the line of sight should not be more than 150 feet.

(6) The line of sight will not be less than 1.5 feet above the ground.

(7) Leveling operations will be conducted under the recommended favorable conditions cited for previous instruments. Also, automatically leveled equipment will not be operated near electromagnetic field generating sources as recommended in previous paragraphs for other instruments. HI of the theodolite and target will be recorded as detailed herein.

(8) Trigonometric leveling as detailed in the paragraphs on EDM operation can be used to determine height differences in lieu of spirit leveling.

(9) The importance of measuring the HI to the accuracies cited cannot be overemphasized. A blunder in the measurement of the HI will transfer through subsequent measurements, reductions, and adjustment, giving inaccurate results. In an effort to measure the HI correctly, proper targets and HI measuring instruments, as well as sound HI measurement procedures, should be followed at all times.

(10) When performing leveling operations, the coefficient of refraction determined for data and applicable for the observation period will be applied.

h. Photogrammetry operations. When performing photogrammetric-based deformation surveys, the metric camera used will be mounted in or on a suitable camera platform (e.g., camera tripod). During exposure, movement of the camera will be minimized. If using an airplane or helicopter for the platform, a camera with an image motion compensator must be used. Typically, 5 to 20 exposure stations are necessary to ensure sufficient precision for the object point coordinates are determined. To ensure the whole photo-taking portion of the survey is performed correctly, it is highly recommended that only experienced personnel be used for this phase of the survey. The photogrammetric reduction process also should be done by experienced personnel trained in image coordinate measurement with the appropriate equipment. If practicable, it is recommended that this process be automated in order to eliminate the possible gross errors possible with self-calibration. EM 1110-1-1000 should be referred to for more specifics on the photogrammetric process.

i. GPS operations. When performing GPS-based deformation surveys, the procedures will be done in accordance with EM 1110-1-1003.

j. Preprocessing of data. Preprocessing of data is done to minimize the amount of subsequent calculations required and eliminate erroneous data.

k. Preprocessing conventional survey data. Preprocessing of conventional survey data consists of applying a rejection test at the time the observations are made in order to reject probable outliers and atmospheric, instrumental, standardization, and geometric corrections so data can be imported directly into subsequent adjustment software. Gravity corrections typically do not need to be applied due to the small areal extent of the data collection. Preprocessing of conventional survey observations can either be done manually or by appropriate verified and validated PC based programs.

l. Preprocessing photogrammetric-based survey data. Preprocessing of photogrammetric-based survey data will include the screening of measured image coordinates in order to reject observations which are outliers and to determine 3-D object coordinates and associated variance-covariance matrix in the local coordinate system. Determination of the 3-D object coordinates should be accomplished by a computer-based bundle adjustment program with self-calibration. Also, in the bundle adjustment, the focal length, horizontal position of the principal point, and coefficients of radial lens, asymmetric lens, and photographic lens distortion and photographic media unflatness will be treated as weighted unknowns. If the distance between exposure is kept to what is recommended, atmospheric refraction also will be neglected. If using GPS observations in conjunction with the photogrammetric process, the appropriate earth curvature correction will need to be applied.

m. Preprocessing GPS survey data. Preprocessing of GPS survey data at a minimum will include determination of the 3-D coordinate differences and associated variance-covariance matrix in the local 3-D coordinate system used for all baseline observed and screening of these reduced vectors to eliminate possible outliers. Table 10-2 lists some rejection criteria for preprocessing deformation survey data, as well as action that can be taken if the data do not pass the rejection criteria.

(1) When a mean uncorrected distance is determined using a steel or invar tape or steel tape, invar tape, or invar wire measuring unit, the following corrections will be applied to determine true distance:

- Temperature correction between the observed and standardized tape distance (ignore if using an invar tape).
- Tension correction between the observed and standardized tape distance.
- Correction due to the unsupported length(s) of the tape.
- Slope correction if applicable.
- Correction due to standardization error.

(2) If the distances collected with an EDM are not rejected, a single uncorrected distance will be computed as the mean of the four independent measured distances. When the mean uncorrected distance is determined, the following corrections will be applied to determine true distance:

- Atmospheric corrections based on the atmospheric conditions during observation and standard atmospheric conditions.
- Correction due to the additive constant for the particular EDM and prism combination.
- Correction due to the standardization error of the EDM.
- Geometric corrections, to include correction due to unequal HI for EDM, prism, theodolite, and target, arc distance to slope distance corrections, slope distance to horizontal distance correction, and horizontal distance at station elevation to horizontal distance at reference elevation correction.

(3) If data collected with an automatic level are not rejected, a single height difference will be computed as the mean of the height difference computed and the height difference computed from the right scale readings. If the foresight and backsight readings are unbalanced, the single height difference will be corrected for vertical collimation error.

n. Analysis of collected data. Prior to analysis of collected data, the adjustment and localization of deformation survey data typically are separated into the 2-D (i.e., horizontal) and 1-D (i.e., vertical) components for analysis. More frequently, though, analysis is being done on 3-D vectors, especially when GPS-based deformation surveys are being used.

(1) The analysis of deformation survey data will include:

- Adjustment of observations made in each epoch in order to determine point coordinates, associated variance-covariance matrices, and other information required for localization of the deformation.
- Localization of deformations between epochs in order to determine statistically significant point movements.

(2) The first step in analysis of the collected data is the survey adjustment. For conventional survey observations, the standard deviations listed in Table 10-3 will be used in the survey data adjustment.

(3) For adjustments where conventional survey data and/or GPS survey observations are combined with photogrammetric survey observations, the localized 3-D coordinates (i.e., *x*, *y*, *z* coordinates) and associated variance-covariance matrix from the photogrammetric survey

Table 10-2
Rejection Criteria For Preprocessing of Deformation Survey Data

Type of Instrument	Type of Measurement	Test	Action to Follow if Data are Rejected
Theodolite[1] or Subtense Bar[1] or Theodolite[1] (Trigonometric Leveling)	Angle Angle Elevation	1. Reduced data must be less than 2 seconds from the mean reduced direction --> Otherwise, reject 2. Reduced zenith angle not being used to compute a height difference must be less than 4 seconds from the mean reduced direction --> Otherwise, reject 3. Reduced and corrected zenith angle not being used to compute a height difference must be less than 2 seconds from the mean reduced and corrected zenith angle --> Otherwise, reject	Re-observe the portion of the survey rejected
Steel or Invar Tape	Distance	1. Difference between two independently measured distances must be less than 2 mm --> Otherwise, reject	Remeasure the distance rejected
Steel Tape, Invar Tape, or Invar Wire Measuring Unit	Distance	1. Difference between two independently measured distances must be less than 0.02 mm --> Otherwise, reject	Remeasure the distance rejected
EDM Distance or EDM Elevation (Trigonometric Leveling)		1. Maximum difference among the four independent measured distances must be less than 5 mm --> Otherwise, reject	Remeasure the distance rejected
Automatic Level Setup	Elevation	1. Difference between readings on the left- and right-hand scale must be within 0.25 mm of rod constant --> Otherwise, reject 2. Difference between height difference determined from the foresight and backsight readings on the left rod scale and that determined from foresight and backsight readings from the right scale must be less than 0.25 mm --> Otherwise, reject	Re-observe the portion of the survey rejected
Network of Level Setups	Elevation	1. Height difference misclosure in a loop must be less than 3 mm * $\sqrt{(K \text{ in km})}$ --> (Minimum = 1 mm) Otherwise, reject	Formulate different loops to determine height differences between points common to loops which have been rejected; or re-observe the portion of the survey rejected
Level and Meter Rule	HI	1. Difference between two independent readings must be less than 0.5 mm --> Otherwise, reject	Remeasure the distance rejected
Mono-comparator, stereo-comparator, or stereo comparator	Photo image coordinates	1. As applied by photogrammetry software for hardware used --> Otherwise, reject 2. Discrepancy between double measured image coordinates is less than 2 microns --> Otherwise, reject	Remeasure image coordinates
GPS Receivers	Horizontal coordinates and elevation	1. Tests as detailed in EM 1110-1-1003	Re-occupy baseline

[1] When performing these data reductions, no atmospheric, instrumental, standardization, and geometric corrections are necessary for angular observation made with a theodolite, except in the case of zenith angles which are observed for the purpose of determining height differences (in which case, earth curvature and refraction need be considered). Because a deformation survey is on a localized network, skew-normal, arc-to-chord, and normal section to geodetic correction need not be applied.

Table 10-3
Standard Deviations to be Used in Deformation Survey Data Adjustment

Measurement	Standard Deviation (σ)[1]
Direction	2 arc seconds
Zenith Angles - when simultaneous or near simultaneous zenith angles are observed and corrected for earth curvature and computed coefficient of refraction	in arc seconds, $\sqrt{(4 + ((61{,}879.5 * S)/(2 * R))^2)}$
Zenith Angles - when 1-way zenith angles are observed and corrected for earth curvature and computed coefficient of refraction	in arc seconds, $\sqrt{(4 + ((123{,}759.0 * S)/(2 * R))^2)}$
Zenith Angles - when 1-way zenith angles are observed and corrected for earth curvature and estimated coefficient of refraction	in arc seconds, $\sqrt{(4 + ((206{,}265 * S)/(2 * R))^2)}$
Distance measured with a steel or invar tape	2 mm
Distance measured with a steel tape, invar tape, or invar wire measuring unit	0.02 mm
Distance measured with a subtense bar	in mm, $\sqrt{(2.25 + (5 * S^2 * 10^{-3})^2)}$
Distance measured with an EDM	in mm, $\sqrt{(25 + (2 * S_s * 10^{-3})^2)}$

[1] σ = estimated standard deviation
S = approximate horizontal distance in km
R = approximate radius of the earth = 6,370 km
S_s = approximate spatial distance in meters

observations and subsequent bundle adjustment and self-calibration will be included. When only using photogrammetric survey observations, the free network constraints (i.e., inner constraints) will be used to define the datum. When photogrammetric survey observations are combined only with conventional survey observation, the datum will be defined by the constraints used in a conventional survey adjustment. When photogrammetric survey observations are combined only with GPS survey observations, the GPS survey observations will be used to define the datum (e.g., location, orientation, scale).

(4) When GPS survey observations are adjusted, they will be adjusted using either the GPS-based localized 3-D coordinates or coordinate differences and associated variance-covariance matrices in accordance with EM 1110-1-1003.

(5) A rigorous form of adjustment (e.g., method of least squares) will be used in the network adjustment software for reducing the deformation survey data. It will be capable of computing the following: the point coordinates and associated variance-covariance matrix, point error ellipses, standardized residuals for each observation, quadratic form of the residuals, total redundancy of the network, and the estimated global variance factor. If only conventional survey observations are to be adjusted in a network, the following additional information is required to be computed: redundancy number of each observation, quadratic form of residuals for each observation type (i.e., directions, zenith angles, etc.), sum of redundancy numbers for each observation type, and the estimated variance factor for each observation type.

(6) The adjustment will take place in two basic steps. The first step is the adjustment of observations and localization of deformations in the reference network. The second is adjustment of observations and localization of deformations in the entire network.

(7) In the first step, all network reference points and all observations directly connecting reference network points will be included in the adjustment. Table 10-4 details the ordinary minimum constraints to be used in the adjustment of conventional survey observations.

If standardized residuals are computed, the observations having values greater than 4 should be examined as possible blunders. Observations identified as blunders will be deleted from the adjustment and the adjustment redone. The estimated global variance factor and estimated

Table 10-4
Minimum Constraints to be Used in the Adjustment of Conventional Survey Observations[1]

Network Type	Minimum Constraint
1-D (i.e., *z*)	*z* of 1 point held fixed
2-D (i.e., *x*, *y*) with distance	*x* and *y* of 1 point held fixed azimuth of 2nd point held fixed (standard deviation of azimuth = 0.1")
2-D (i.e., *x*, *y*) without distance	*x* and *y* of 2 points held fixed
3-D (i.e., *x*, *y*, *z*) with distance	*x*, *y*, and *z* of 1 point held fixed azimuth and zenith angle to 2nd point held fixed zenith angle to 3rd point held fixed (standard deviation of azimuth and zenith angles = 0.1")
3-D (i.e., *x*, *y*, *z*) without distance	*x*, *y*, *z* of 3 points held fixed

[1] *x* = *x* horizontal value
y = *y* horizontal value
z = *z* vertical value (i.e., elevation)

NOTE: 2-D and 3-D network minimum constraints shall be applied to opposite sides of the network.

variance factors for each observation type for the adjustment should be near 1. If they are less than 0.5 or greater than 2.0, all of the input variances should be rescaled by a ratio of the estimated variance factor of observation type divided by the estimated global variance factor and the adjustment rerun. All epochs of observation should then be adjusted in the same manner with regard to the constraints applied, standardized residuals and scale of the input variances.

(8) The results will be examined to ensure the reference points are stable between epochs. Any reference points which are not found to be stable will be excluded from the reference network.

(9) For the second step involving adjustment of observations and localization of deformations in the entire network, the constraints will be either (a) all reference network points found to be stable are held fixed or (b) all reference network points found to be stable forming a free network of points. The first option will result in a very constrained adjustment. Examination of the standardized residuals and rescaling the input variances for each epoch of observation will be done as described in paragraph (7) above.

(10) For localization of deformations in the entire network under step two, the same constraints shall be applied for the epochs being analyzed. The tests detailed herein will be applied. The final results of this step are apparent movements of all object points and any excluded reference points and indications of whether or not these point movements are statistically significant.

(11) The localization and subsequent tests described are being applied to single points to determine their possible movement. Even though this is the case, such localization and testing can be applied to movements of groups of points if a priori information indicating they may have moved is available.

10-14. Final Reports

A final report will be required for each deformation survey or project done where results have been determined. Any deviations from the specifications detailed in this manual will be included in this report.

a. The final report will include figures with plan or cross-sectional views showing the outline of the structure, location of deformation network points and their names. All reference points shown in the figures will be denoted by one symbol, while all object points will be denoted by a different symbol. Point movements will be plotted as vectors with their associated error bars and/or error ellipses. Statistically significant movements will be flagged. Only displacements between two chosen epochs will be plotted on a given figure. Displacement contours will not be plotted.

b. The final report will include a tabular summary of each network adjustment and at a minimum the following information relative to the adjustments made:

- The constraint applied.
- The names of points used.
- The adjusted point coordinates to the nearest 0.1 mm.
- The standard deviations of point coordinates to the nearest 0.1 mm.

- The dimensions of error bars to the nearest 0.1 mm at the one standard deviation level for 1-D network points, dimensions of the axes of the error ellipses to the nearest 0.1 mm at the one standard deviation level plus orientation angle to the nearest 0.1 degree for 2-D network points, and dimensions of the axes of the error ellipses to the nearest 0.1 mm plus out-of-plane angles to the nearest 0.1 degree for 3-D network point coordinates.

- The quadratic form of the residuals.

- The total redundancy of the network.

- The estimated global variance factor.

If the network adjustments use only conventional survey observations, in addition to the above, the following will be required in the final report:

- The redundancy number for each observation.

- The standardized residuals for each observation.

- The quadratic form of residuals for each observation type (e.g., directions, zenith angles, distance, etc.).

- The sum of redundancy numbers for each observation type.

- The estimated variance for each observation type.

c. The final report will include another summary table of each localization of deformations. Such a table will include:

- The constraints applied in the two network adjustments.

- The names of points used.

- The apparent displacements to the nearest 1 mm and associated direction to the nearest 0.1 degree.

- The dimensions of error bars to the nearest 0.1 mm at the one standard deviation level for 1-D network point displacements, dimensions of the axes of the error ellipses to the nearest 0.1 mm at the one standard deviation level plus orientation angle to the nearest 0.1 degree for 2-D network point displacements, and dimensions of the axes of the error ellipses to the nearest 0.1 mm plus out-of-plane angles to the nearest 0.1 degree for 3-D network point displacements.

d. The final report will include figures showing 1-D cumulative displacements of critical points in critical directions versus time. Examples of critical cumulative displacements include movements in the downstream and vertical directions of a small number of points on the crest of a dam or movements in the downhill and vertical directions of a small number of representative points in a earthen dam or levee. The error bar associated with each displacement will be plotted with these displacements. Data from all deformation analyses performed on the project will be included. Statistically significant cumulative displacements will be flagged.

Chapter 11
Deformation Monitoring

11-1. General

This chapter provides guidance for performing field measurements in support of the RLR technique of periodic structural monitoring. This method provides a direct measurement of displacement as a function of time, and has fewer problems of evaluation than most other types of instrumentations. The Robertson method is broken down into two parts: the initial setup of the control network, and the periodic measurement to determine movement. An EDM or total station and trilateration techniques are used for this type of deformation monitoring.

11-2. DME

All modern DME measure distance by timing, in an indirect fashion, how long it takes light to make a round trip to a reflector. By knowing the velocity of light, the distance may be calculated from 2d = vt, where d is the distance to the reflector, v is the velocity of light, and t is the time required for light to travel to the reflector and back. Light travels 1 foot in approximately a nanosecond. Therefore, other methods must be used to avoid the problems of timing a pulse to a small fraction of a nanosecond. In surveying a continuously operating source of light is used, and this light is modulated in a known way (i.e., the light is turned off and on in a regular fashion). The modulation wavelength λ_m is determined by the rate at which the light is modulated and by the velocity at which the light is traveling. A measurement is then made of the phase difference between the light proceeding toward the reflector and that returning. In an instrument built with a 20-m-modulation wavelength, the same result would be obtained every 10 m. For example, if an answer of 3.462 m resulted from a measurement (with a 20-m-modulation wavelength), the operator would not know if the complete answer would be 3.462, 13.462, 23.462, or 2,183.462 m. It would then be necessary to switch to a longer modulation wavelength. If the new wavelength were 10 times longer, the instrument would be able to determine the correct value of the figure in the tens place. To determine the figure in the hundreds place, the wavelength would again be increased ten times. This procedure would be repeated until the entire distance was resolved. Some instruments require five or six modulation frequencies to resolve ambiguities completely. By using the phase comparison method, instruments can be built that measure to better than one-thousandth of a modulation wavelength. Thus, certain instruments have resolutions finer than 0.001 m.

11-3. DME Error Sources

This section refers specifically to error sources, which is an inherent property of the instrument itself and is in addition to external factors, such as plumbing and measurements of refractive index. These systematic errors are usually written as (a) mm + (b) mm/km, where (a) and (b) are maximum values for a particular instrument.

a. Constant instrument error. The (a) portion of the error is a combination of several small errors, which are independent of the length of the line being measured. The more important of these are:

(1) Instrument resolution. Resolution is a property of the instrument that results from its original design. In the case of DME, it might be defined as the smallest change in distance to a target that causes a corresponding change in the reading obtained from the instrument. The resolution can be no finer than the scale or digital display can read. However, a display that can be read to a millimeter is no assurance that the resolution of the instrument is also a millimeter.

(2) Cyclic or delay line error. If the manner in which the light is modulated distorts the sinusoidal pattern of the outgoing beam or if the phase comparison technique of measuring the returning beam is less than perfect, a cyclic error will occur. The error is named cyclic because it repeats itself every modulation wavelength. If the effective modulation wavelength of a particular instrument is 10 m and the cyclic error for a measurement of 6 m is 4 mm, then the cyclic error at 16, 26, and 36 m would also be 4 mm. At the same time, the instrument might have zero cyclic error at 1, 11, and 21 m. A determination of cyclic error consists of making comparative measurements throughout a modulation wavelength.

(3) Instrument-reflector calibration. When a DME is received from the manufacturer, one or more reflectors are usually received at the same time, and these have been assigned a calibration constant by the manufacturer. This constant is to be added or subtracted from a distance reading in order to obtain a correct distance. In most cases, the constant is sufficiently accurate for routine work. However, for greater precision or for reflectors that are obtained from other sources, it is necessary to determine accurately the constant of each reflector. Figure 11-1 shows two reflectors, both of which are mounted

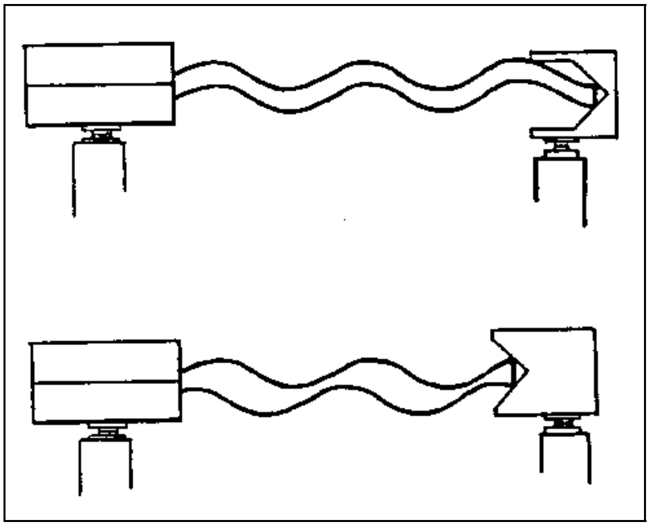


Figure 11-1. Reflector calibration

at the same distance from the measuring instrument. Because the reflectors are at different positions with their mounts, they give different readings for the same distance. The difference between the true distance and the reading obtained with a particular reflector is the instrument-reflector constant.

(4) Offset. The reflector calibration includes two sources of error. The first error is caused by the reflector not being optically above the point over which it is plumbed. The second error is caused by the DME electrical center, the point from which the instrument measures, not being over the point over which it is plumbed. However, both errors are corrected when the instrument-reflector combination is calibrated. On the other hand, if the electrical center of the instrument should shift as the electronics age, the instrument-reflector constant would no longer compensate for this shift, and an offset error would result. Even so, it is possible to measure the magnitude of the offset error. At weekly intervals, simply measure a short line (100 m) using the same reflector (the line should be outside so that the instrument will be subject to a variety of temperatures). If each measurement is made using the same procedures, the differences in length in excess of the resolution error are due to changes in the offset or electrical center of the DME. Temperature and pressure corrections must be made to these measurements. Offset changes of 5 mm may occur in some instruments.

(5) Pointing error. The modulation wave front issuing from a properly designed and operating instrument is at all points equidistant from the center of the instrument (Figure 11-2). It might be likened to the waves around a stone dropped into water. However, the wave front may be distorted in passing through the modulator, and then a portion of the wave may be ahead or behind the remainder. In Figure 11-2, the instrument sees both reflectors as equidistant because the phase of the modulated wave is the same for both. If the instrument is moved in azimuth slightly, the distance that is read would change. This type of error may be detected simply by multiple pointing at a reflector. If different pointings yield different results, it may be necessary to take several readings in the field, swinging off the target and then back until two or three sets of readings agree well. Practice in the field may help eliminate this problem as an experienced operator tends to point and adjust an instrument in the same way for each measurement.

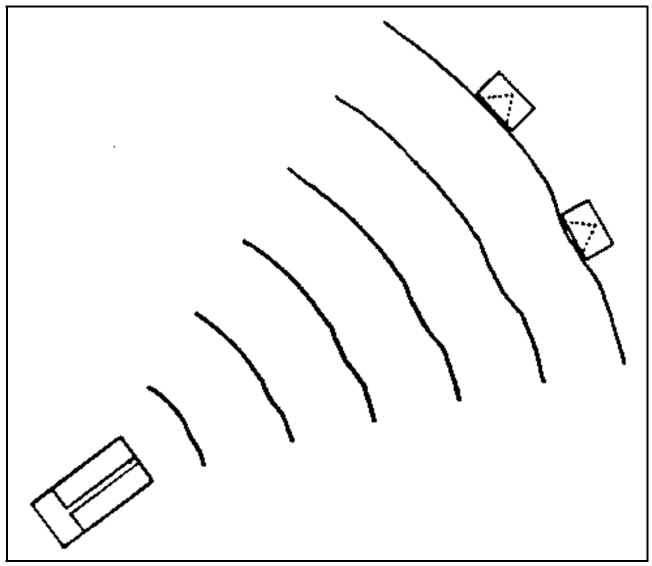


Figure 11-2. Pointing error

(6) Total error. The previous sections have treated individually the various types of errors in the (a) portion that may be present in DME. However, the total error (a) is all that is desired.

b. Measurement dependent errors. The second (b) portion of the error is due to the short- and long-term variations in the frequency standard used to control the modulation frequency. This is usually a quartz crystal, which may or may not be mounted with a small oven for temperature control. The only proper method of checking this frequency is by using a properly calibrated electronic counter. Some manufacturers provide a connector on the instrument so that the frequency may be easily monitored.

11-4. Atmospheric Corrections

a. Refractive index. The accuracy of measurements made with DME depends not only upon the instrument itself but also upon a knowledge of the velocity of light along the measuring path. Techniques to be discussed later reduce this dependence to a minimum in the case of measuring movements in large structures, but in order to make absolute measurements, it is vital to have an accurate knowledge of the refractive index of the air along the line being measured. The refractive index is simply a measure of how much light is slowed down in traveling through a medium other than a vacuum. Some representative refractive indices used with modern DME are given in the following table.

Wavelengths (micrometers)	Source	Refractive Index
0.48	Xenon arc	1.0003101
0.63	HeNe laser	1.0003002
0.84	GaAs laser	1.0002947
0.91	GaAs diode (infrared)	1.0002936

These refractive indices are at standard conditions (temperature °C, pressure 760 mm of mercury, Hg). For any other conditions of temperature and pressure, a new refractive index may be found from

$$n_a = 1+[(n_g-1)/(1+T/273.2)]*(P/760)$$

where

T = temperature, °C

P = pressure, mm of Hg

Generally the atmospheric corrections are supplied in the form of a handy circular slide rule, tables, or a simple nomogram, which gives a parts-per-million correction depending on temperature and pressure. This correction may be either dialed into the instrument or applied directly to the measured distance. The procedure in most cases would be to measure both temperature and pressure at each end of the line. The temperatures and pressures must then be meaned, and parts-per-million correction must be determined from the appropriate source. Then, this correction must be dialed into the instrument before a measurement can be made. In the case of work on dams, this procedure could prove to be awkward. In most cases, it would be simpler to set the parts-per-million dial to zero and keep a record of temperature and pressure to be applied later as a correction.

b. Measurement of temperature and pressure. When absolute accuracy in measuring is required, such as when a baseline is laid out, temperature and pressure measurements play a vital part. The magnitude of the errors possible from the incorrect application of temperature and pressure corrections are: a change of 1 °C or a change of 2.5 mm (0.1 inch) of Hg will cause a 1 ppm change in the observed distance.

(1) Pressure measurements should be made at both ends of the line, and the mean of the two values used in the refractive index equation. If it is not possible to place barometers at both ends of the line, place the barometer at the instrument end, and use the elevations of the two ends together with the pressure measured at the instrument to calculate the pressure at the other end.

(2) Temperature is much more difficult to measure properly. The measuring equipment must be well shielded from the sun's radiation. This can be accomplished by enclosing the thermometer in a reflective insulating shield. However, this permits the heat to build up within the shield, and thus a small fan or some other means must be used to move air over the temperature sensing device so that the true air temperature is read. Also, temperatures measured at the end points of a line near the ground are a poor indication of the true temperature along the line. Studies have shown that during the day, temperatures near the ground are much warmer than those 30 m above the ground. A 5 ºC difference is not uncommon. At night the reverse is true; temperatures near the ground are cooler than those above. Unfortunately, many lines to be measured in deformation surveys are more than 30 m above the ground over most of their lengths. Thus, temperature measurements are one of the major sources of error in the accurate determination of distance.

(3) In measuring dams or other large structures, refractive index errors are less important because displacement values are needed; therefore relative rather than absolute distances may be used.

11-5. Ratios

Index errors (temperature and pressure measuring errors) have been shown to limit the accuracy of DME. However, even when temperature and pressure are measured properly, errors still occur because it is difficult or impossible to measure other than at the end points of the line. Further, refractive index measurements are both time consuming and expensive. This section discusses techniques for reducing refractive index errors in measurements of large structures by using ratios, or reference lines. There are two important rules when using DME:

Rule 1: Refractive index errors, resulting from end point measurements of temperature and pressure, tend to be the same for all lines measured from one point within a short period of time.

Rule 2: The ratios of observed distances, measured from one point within a short period of time, are constant.

Note: For both rules, a short period of time is 30 minutes or less.

a. In Figure 11-3, lines AB and AC are measured from a common point. Rule 1 states that if refractive index measurements are made at points A, B, and C within a short period, the errors in the measurements tend to be the same at all three points. If the true temperature along line AB is 20 °C, but the mean of measurements made at A and B is 24 °C (a condition typical of daytime), then the mean of temperature measurements at the end points of line AC would also be expected to be 4 °C higher than the true temperature along that line. Because 1 °C is approximately equivalent to 1 ppm of distance, both lengths will be in error by 4 ppm. However, if the measured length of AB is divided by the measured length of AC, the resulting ratio will equal the ratio of the true lengths. Thus, the *ratio* of two measured lengths will be more accurate than either of the lengths that were used to form the ratio. For example, AB was measured to be 2,839.611 m, and AC was measured to be 2,241.487 m. Their ratio is AB/AC = 1.26684250. Both lines were in error by 4 ppm because of temperature-measuring errors; therefore, the true lengths were AB = 2,839.611 + 0.0114 (4 ppm) and AC = 2,241.487 + 0.0090 (4 ppm). The ratio of the true lengths is 2,839.6224/2,241.4960 = 1.26684250, the same as the ratio of measured lengths.

b. When ratios are formed from measurements that have had refractive index corrections applied, they will be called *corrected ratios*. The property of the corrected ratio is that it is very accurate. From corrected ratios, angles may be computed that are frequently within a few tenths of an arc second of their true values.

c. A second set of ratios can be obtained from the same measurements by using the data before the application of the refractive index corrections. These are called *observed ratios*, and they have been formed from lines that have had no temperature or pressure corrections applied. Rule 2 states that the observed ratio is constant. This means that the observed ratio of two lines measured today will agree with the observed ratio of the same two lines measured months or years later. This will be true even though the observed lengths of the individual lines have changed greatly because of changes in atmospheric conditions between the two sets of measurements. The observed ratios will not, however, be the same as the corrected ratios unless certain conditions are met. To understand this, let us assume for a moment that an instrument has been set upon a hilltop. In the valley below, two points have been selected that are equidistant from the hilltop stations and are at the same elevation. The observed distances to the two points would appear the same because the distances are equal and both lines pass through roughly the same atmosphere. A point is then selected that is the same distance from the hilltop station as the other points, but with a higher elevation. When the observed distances are recorded, the two lengths to the valley points are the same, but the observed length to the higher elevation point is shorter. Because air density decreases with elevation, the light traversing the higher line travels faster and returns sooner. The instrument then shows the distance to be shorter. Two lessons can be learned from this. The first lesson is that if the mean elevations of two lines measured from a point are the same, the ratio of the observed distances is equal to the ratio of the corrected distances. In the example above, the observed distances to the valley points are the same, and the ratio of the two observed lengths is 1. The true lengths to the two points are the same so that the ratio of the corrected lengths is also 1. This is often the case with dams where the alignment markers along the

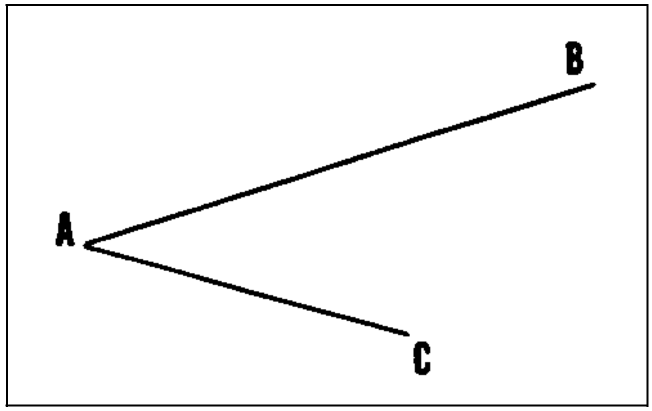


Figure 11-3. Ratio of two lines

crest of the dam are all within a few meters of the same elevation. This property of *observed ratios* will be used later on.

d. The second lesson is that when the elevations of the end points to which measurements are being made are different, the ratio of observed lengths is not the same as the ratio of corrected (true) lengths because the refractive indices along the two lines are different. Even though it is not accurate, the observed ratio does not change with time and it may be used to detect changes in position. Furthermore, the observed ratio may be corrected by means of an atmospheric model.

e. In many respects, ratios have properties similar to those of angles. In triangulation, the sum of the three angles of a triangle must equal 180, and a knowledge of two angles permits calculation of the third. Similarly, the product of three ratios obtained from a triangle must equal 1, and a knowledge of two ratios permits calculation of the third. In Figure 11-4, the triangle shown has sides A, B, and C as measured from vertices 1, 2, and 3. The ratio measured from vertex 1 is A_1/B_1, using a counter-clockwise convention (A_1/B_1 rather than B_1/A_1) with the subscript designating the vertex from which the ratio was measured. Two other ratios, B_2/C_2 and C_3/A_3, may also be measured. If the measurements are perfect, $A_1 = A_3$, $B_1 = B_2$, $C_2 = C_3$, and $A_1/B_1 * B_2/C_2 * C_3/A_3 = 1$. If the measurements are not perfect (the usual case), the degree to which the product failed to equal 1 is a measure of the precision of the measurements. If only two ratios were measured, the third may be calculated. For example, $A_1/B_1 = C_2/B_2 * A_3/C_3$. Angles may be calculated directly from the ratios by using a modified cosine formula.

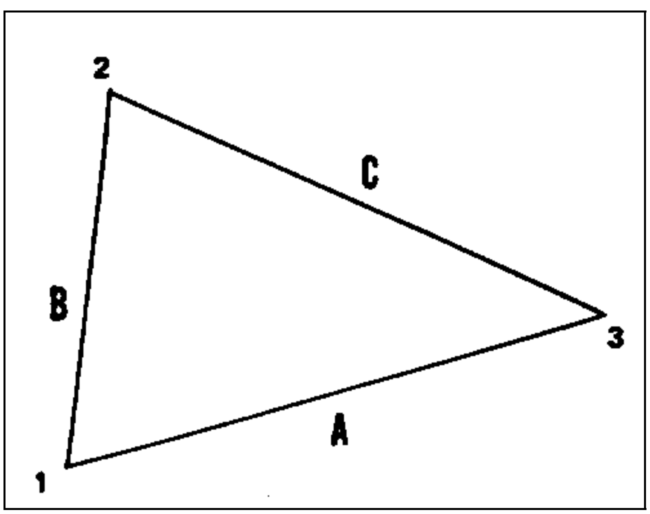


Figure 11-4. Ratios in a triangle

f. The use of ratios yields angles as a result, and the angles determined from the ratios are more accurate than those determined from the lengths alone because a ratio is more accurate than either of the lengths from which it is derived.

g. When the angles of a triangle do not sum to 180, the triangle may be adjusted by taking one-third of the difference between 180 and the sum of the angles and by applying it as a correction to each angle. With ratios, a correction may be made to each ratio.

11-6. 2-D Deformation Monitoring

Sometimes it is not practical to do First- or Second-Order 3-D deformation monitoring. Measurements of movements in large structures can be made very accurately, in two dimensions, by using trilateration techniques. The work consists of two phases, the control network and the structure itself.

a. The control network. In monitoring possible movements of structures, points on the structure must be related to points that have been selected for stability, usually at some distance from the structure itself. These will be called control points, and all movements of the structure will be related to one or more of them. It is important that the control points not move, and for this reason, they should be placed in geological stable positions. They should also afford a good geometry for trilateration measurements. Good geometry, in turn, consists of measuring along the line where movement is expected. For example, if measurements of upstream or downstream movements are required, the control point should be located either upstream or downstream. Further, the point should be at a sufficient distance from the structure so that the end points, as well as the center, can be monitored with good geometry. In Figure 11-5, a dam is shown with both an upstream and a downstream control monument. Geometrically, measurements from the upstream side of the dam will be poor, while those from the downstream side will be much stronger. If movement in two dimensions is desired, a point off the end of the dam should also be chosen (see Figure 11-5). For best results, the angle of intersection (θ) should be 90 degrees. Two selections of control figures are shown in Figure 11-6.

(1) A final criterion for the selection of control monuments is intervisibility. Because the control figure also provides a means of correcting for refractive index, the points selected for control at the ends of the dam must be visible from the upstream and/or downstream points.

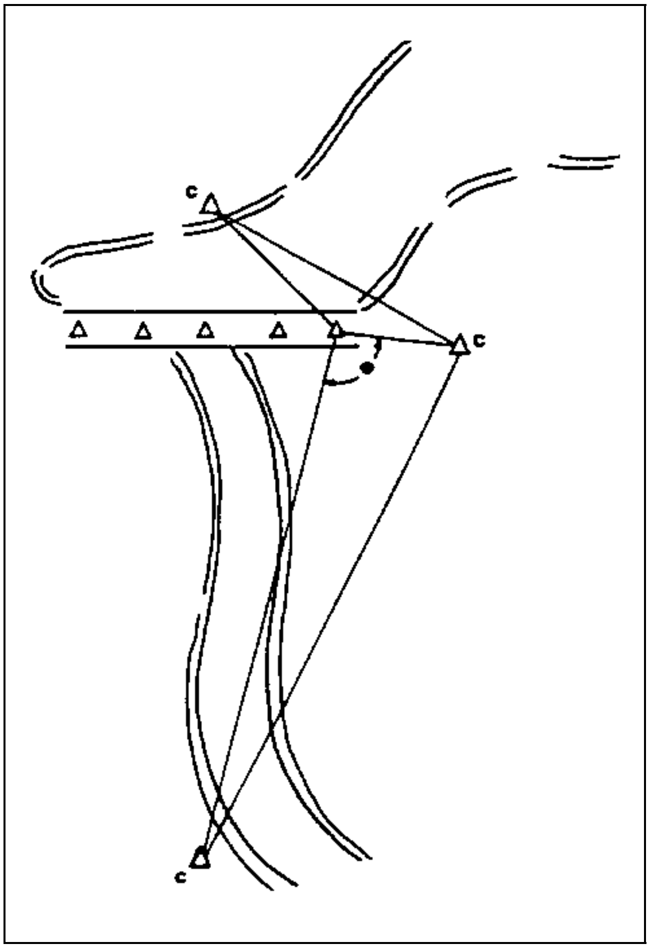


Figure 11-5. Control monuments (c) for a dam

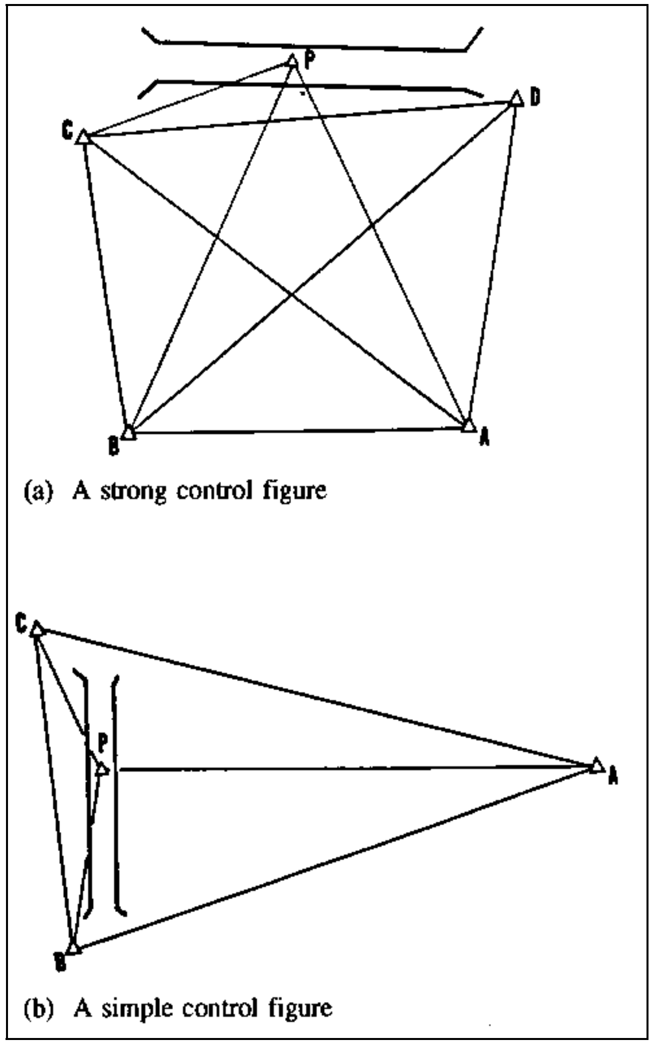


Figure 11-6. Strong and simple control figures

(2) In trilateration, lengths to an unknown station from each of two control points will give the position of the unknown station in two dimensions. Measurements from three control stations will give three positions of the unknown station, and may be used as a check of survey accuracy. Figure 11-6a shows a good control figure for the measurement of a dam. In the control figure, A, B, C, and D are control monuments. All are intervisible. Point P is an unknown station on the dam and is measured from control points A, B, and C. Positions of P are calculated from measurements of lines AP and BP, from lines BP and CP, and from lines AP and CP. The agreement between the three positions obtained for point P is a measure of the accuracy of the survey.

(3) When measurements are made of lines exceeding 600 m, a major source of error is the inability to determine accurately the refractive index along the line. An error in temperature of 1 °C or in pressure of 2.5 mm (0.1 inch) of H_g will cause an error in length of 1 ppm. These errors may be minimized by considering the ratio of two lines that have been measured within 30 minutes of each other. The errors of each line tend to be the same so that taking a ratio greatly reduces the magnitude of the error. This may be shown by again referring to Figure 11-6a. Point P has been selected as a reference point. Its position was chosen so that it would be in stable ground, it would be visible from the other control points, and the lines to it from the other control points would pass through atmospheric conditions similar to those from the control points to unknown positions on the dam.

(4) The first time a dam is visited to make trilateration measurements, both ratios and conventional measurements are made to determine the shape and size of the

control figure. The simplest example would be the triangular figure shown in Figure 11-6b. All control monuments should be occupied by the DME. At each point, measurements should be made to all of the other control monuments within a short period of time. In the case of the triangle ABC in Figure 11-6b, monument A would be occupied and lengths AC and AB measured. Careful refractive index readings should also be taken at both ends of each line as it is measured. Similar measurements should then be made as the DME occupies stations B and C. Each line should then be reduced to the level of spheroid and have the refractive index corrections applied. A typical set of measurements for triangle ABC is:

	Length (in m)	Ratio
A to	C 2,547.447	AC/AB 0.9181349
	B 2,774.589	
B to	A 2,774.583	BA/BC 3.7776155
	C 734.480	
C to	B 734.478	CB/CA 0.2883212
	A 2,547.430	

(AC/AB)*(BA/BC)*(CB/CA) = 1.0000018

Adjusted Angles

A	15°	05'	47.84"
B	64°	35'	55.08"
C	100°	18'	17.08"

By way of comparison, angles calculated from the mean lengths would be:

A	15°	05'	47.59"
B	64°	35'	53.76"
C	100°	18'	18.65"

Note: lengths are spheroid distances in meters.

The adjusted angles determined from corrected ratios are more accurate than the angles determined from the means of the lengths of the sides because ratios are more accurate than the lengths of which they are composed.

(5) It may be seen from the above example that the result of working with ratios is angles, and that in effect very accurate triangulation is being carried out using DME. As in the case of triangulation, a base line is necessary to determine the scale when ratios are used. Choose one of the sides of the triangle to serve as a baseline, and use the mean length as the scale for the triangle. In this example, AB has been chosen and its length is 2,774.586 m. Next, by using the sine formula and the angles determined from ratios, the other two sides may be determined:

2,774.586/sin C = BC/sin A = AC/sin B

BC = 734.481 AC = 2,547.443

(6) The angles obtained by these methods are of the highest accuracy. The scale, however, is only as accurate as the mean of the two measurements of the baseline. Fortunately, this is not a serious problem with measurements of dams because changes in lengths are desired rather than the absolute lengths themselves.

(7) The final task in establishing the control network is to assign coordinates to A, B, and C. These may be fitted into an existing network, or a local control net may be set up for the project.

(8) At a later date, the control figure may once again be occupied. The same procedure may be used, and the angles determined and compared with those obtained during the first survey. This, however, requires the use of temperature and pressure measuring devices each time the figure is surveyed.

(9) NOTE: An easier method is to use the observed ratios, for these do not require knowledge of the refractive index. Remember that the observed ratios remain constant, and thus comparison of observed ratios from the first survey with observed ratios from the second survey is sufficient to determine whether any of the control monuments have moved. In fact, measurements of temperature and pressure need only be made of the control lines in order to give the proper scale to the figure. And these measurements need only be made the first time a project is surveyed. From that time on, only observed distances are required. In addition, all of the measurements from the control monuments to stations on the dam will be observed distances. Measurements of temperature and pressure are not necessary.

b. Points on the dam. When positions have been established for the monuments in the control figure, observed ratios will be used to determine the refractive index corrections for measurements of points on the dam. Referring again to Figure 11-6b, the lines AC, AB, and BC have been corrected for refractive index and may be used as reference lines. For measurements from control monument A, either AC or AB may be used as a reference line. A good reference line is one which traverses approximately the same atmosphere as is found along the lines to points on the dam and is almost the same length or longer. If we call the corrected length of the reference line R_{corr} and the observed length of the same line R_{obs}, the following equation may be written

$$R_{obs} \times k = R_{corr}$$

Where k is a constant owing to the atmospheric conditions along the line at the time it was measured. Because the reference line has been selected to travel through approximately the same atmosphere as that to points on the dam, we may say that k is also the atmospheric constant for lines measured to the dam. If P_{obs} is the observed length to a point on the dam, then the corrected distant, P_{corr}, may be found from

$$P_{obs} \times k = P_{corr}$$

This technique enables the surveyor to correct for refractive index without using temperature and pressure measuring equipment. However, k is not really a constant. It changes slowly with time. For this reason, it must be remeasured at approximately 30-minute intervals, and it must be assumed it changes in a linear fashion.

(1) The following example will detail the previous phenomena. In Figure 11-7, the DME has been set up at A. Measurements are made of AC, AP_1, AP_2, AP_3, and again AC.

After the observed lengths have been reduced to the level or the spheroid, the measurements from control monument A were recorded as listed in Table 11-1.

The first and last measurements are of AC. The length of AC is known and is used as a reference line to calculate the value of the refractive index constant. At first, the constant was 1.0000459 (2,547.443/2,547.326), but because of changes in the atmosphere, it changed to 1.0000440 (2,547.443/2,547.331). The value of k at intermediate times may be found by assuming that the change was linear. Thus, a value of k may be found for the times when P_1, P_2, and P_3 were measured. Applying the appropriate value of k to the observed length, D_{obs}, of AP_1 gives 2,477.075 * 1.0000454 = 2,477.187 as its corrected length, D_{corr}.

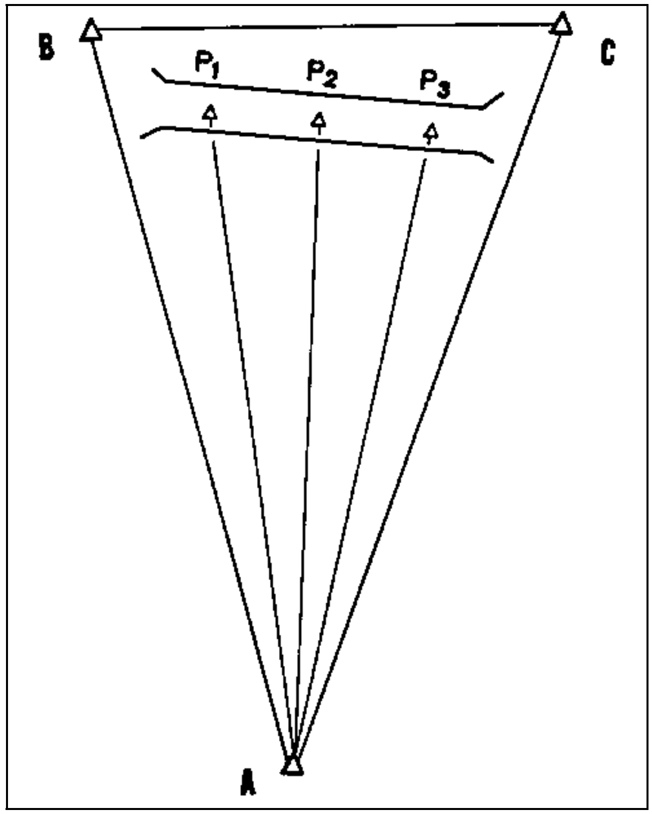


Figure 11-7. Use of a reference line

Table 11-1
Measurement Taken (Example Deformation Survey)

To Station	Time	Observed Length (D_{obs})	Refractive Index Constant (k)	Corrected Distance (D_{corr})
C	1330	2,547.326	1.0000459	2,547.443*
P_1	1335	2,477.075	1.0000454	2,477.187
P_2	1340	2,407.354	1.0000449	2,407.462
P_3	1345	2,445.152	1.0000445	2,445.261
C	1350	2,547.331	1.0000440	*2,547.443

*Note: AC is the reference line

(2) Any length in a control figure may serve as a reference line, although some lines will be better than others. From A, AB would also serve. From B however, BC would be a better choice than BA because it passes through an atmosphere similar to that found in measuring from B to P_1, P_2, and P_3.

c. Reduction to the spheroid. Mention has been made of reducing lines either to the level or the spheroid. In very accurate work where lines exceed 1 km, the surface upon which a survey is being made can no longer be considered a plane. If distances are reduced to the level and used to calculate angles, the angles thus obtained may not agree with angles obtained from a theodolite. Further, the position of a point calculated from the lengths to two control monuments may not agree with the position of the same point when measured from two other control monuments. To prevent problems of this type, figures with line lengths in excess of 1 km should be reduced to the spheroid instead of the level. The equation to be used is

$$D_s = R \sqrt{\frac{[D_1 - (e_2 - e_1)]\ [D_1 + (e_2 - e_1)]}{(R + e_2)\ (R + e_1)}}$$

where

D_s = Spheriod chord distance

R = Earth radius (6,372,000 meters)

D_1 = Observed distance from the DME

e_1 = Elevation + H.I. of the instrument

e_2 = Elevation + H.I. of the reflector

d. RLR deformation survey example. The example survey developed in the following paragraphs combines the principles developed for RLR method of deformation monitoring. This section will combine these elements to show how they may be used for a precise survey of a dam.

(1) A diagram of the control setup and dam are shown in Figure 11-8. Control pedestals have been set at points C1, C2, C3, and C4. Markers A1 through A6 have been set along the crest of the dam, and T1 and T2 have been set near the toe of the dam. Elevations have been measured to obtain the list in Table 11-2.

(2) Each of the control monuments were occupied with an EDM, and measurements were made to the other three control monuments. Temperatures and pressures were also taken at both ends of the lines. After measuring the control lines, the lengths to stations on the dam were measured from three of the control monuments. Temperatures and pressures were not taken for these lines.

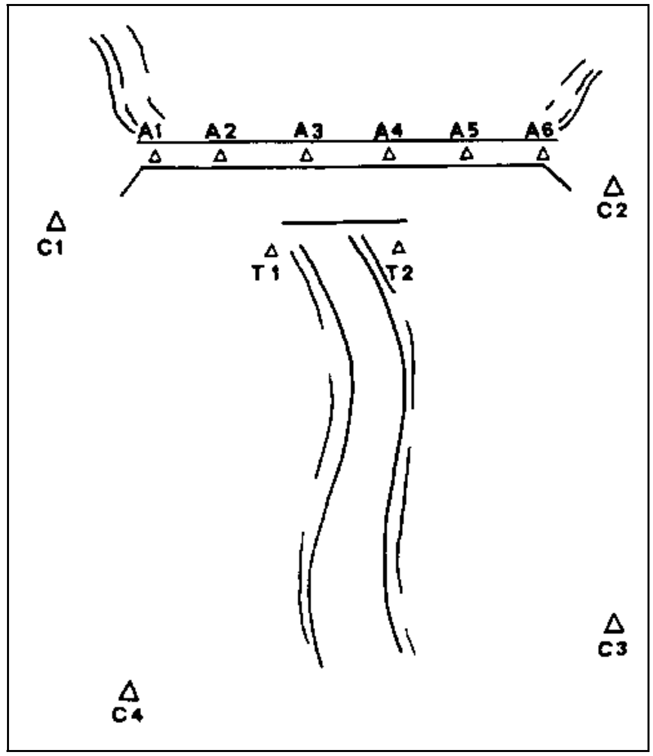


Figure 11-8. Fictitious dam

Table 11-2
Elevations for Example Deformation Survey

Point	Elevation (m above sea level)
A1	410.724
A2	410.718
A3	410.706
A4	410.721
A5	410.712
A6	411.245
C1	419.911
C2	413.275
C3	463.701
C4	521.537
T1	329.623
T2	329.394

(3) On a separate occasion, the following lengths were measured from C3 (Table 11-3).

Measurements began with the control figure. Either C1 or C2 could have been used for a reference line, but in this case C1 has been chosen. Because it was the reference line, it was measured before and after the remaining control lines. This practice helped to check for both drift

Table 11-3
Measurements from C3 for Example Deformation Survey

#	To	Time	Observed Distance D_S (meters)	Mean Temp. (°C)	Mean Press. (inches Hg)
1	C1	0930	1,081.105	16.4	28.26
2	C4	0935	945.032	16.4	28.09
3	C2	0940	703.788	17.0	28.27
4	C1	0945	1,081.104	16.7	28.26
5	C1	1025	1,081.101		
6	A1	1035	968.241		
7	A2	1045	924.456		
8	C1	1050	1,081.103		
9	A3	1100	882.721		
10	A4	1115	843.323		
11	C1	1120	1,081.104		
12	A5	1130	806.626		
13	A6	1145	772.950		
14	C1	1150	1,081.104		
15	C1	1300	1,081.100		
16	T1	1305	872.886		
17	T2	1315	836.021		
18	C1	1300	1,081.097		

in the instrument and in the atmospheric conditions. When the control lines were completed, the operator next measured to points on the dam. Forty minutes had elapsed after completion of the contour line measurements before the field party with reflectors were set up on the dam. Because the reference line should be measured approximately every 30 minutes, the observed distance to C1 was again measured (measurement 5). A reflector was left unattended at C1 because it was no longer necessary to read the temperature and pressure. Remember temperature and pressure measurements are made only on the control lines and only when a study is made for the first time at a particular dam. The next time the dam is visited, perhaps 6 or 12 months later, it will not be necessary to measure refractive index. Possible movement in the control figure may be checked at that time by a comparison of ratios of observed distances.

(4) Measurements were made that same afternoon from C1. Only the control lines were measured. Three sets of positions will be obtained for the stations on the dam from C2, C3, and C4. Measurements from C1 would do little to improve the accuracy of these positions in the upstream-direction. Table 11-4 gives the lengths from C1 recorded for that session.

(5) A week later, monument C4 was occupied and measurements were taken. These measurements are shown in Table 11-5.

Table 11-4
Measurements from C1 for Example Deformation Survey

#	To	Time	Observed Distance D_S (meters)	Mean Temp. (°C)	Mean Press. (inches Hg)
19	C2	1400	566.212	19.0	28.28
20	C3	1405	1,081.095	18.8	28.20
21	C4	1410	989.418	18.5	28.09
22	C2	1415	566.215	18.8	28.28

Table 11-5
Measurements from C4 for Example Deformation Survey

#	To	Time	Observed Distance D_S (meters)	Mean Temp. (°C)	Mean Press. (inches Hg)
23	C1	0835	989.446	6.1	28.85
24	C2	0840	1,138.277	6.1	28.87
25	C3	0845	945.050	5.8	28.78
26	C1	0850	989.445	6.2	28.85
27	C1	0900	989.444		
28	A1	0905	1,031.587		
29	A2	0915	1,042.973		
30	A3	0925	1,057.756		
31	C1	0930	989.438		
32	A4	0940	1,075.788		
33	A5	0945	1,096.925		
34	A6	0955	1,120.924		
35	C1	1000	989.432		
36	T1	1010	981.303		
37	T2	1020	987.682		
38	C1	1025	989.431		

(6) Later that day, monument C2 was occupied and measurements were taken. These measurements are shown in Table 11-6 and completed the field measurement phase.

Table 11-6
Measurements from C2 for Example Deformation Survey

#	To	Time	Observed Distance D_S (meters)	Mean Temp. (°C)	Mean Press. (inches Hg)
39	C1	1230	566.225	8.1	29.04
40	C4	1235	1,138.273	7.6	28.87
41	C3	1240	703.799	7.8	28.97
42	C1	1245	566.225	8.3	29.04
43	A1	1250	398.146		
44	A2	1300	337.350		
45	A3	1310	276.652		
46	C1	1315	566.225		
47	A4	1320	216.070		
48	A5	1330	155.828		
49	A6	1335	96.436		
50	C1	1345	566.224		

(7) The first step in the data reduction is to reduce all the lines (D_S) to the spheroid. This has been done and is shown in Table 11-7.

Table 11-7
Corrected Line Lengths

#	C3 To	Time	Observed Distance D_{obs} (meters)	Corrected Distance (meters)
1	C1	0930	1,080.143	1,080.156*
2	C4	0935	943.188	943.201*
3	C2	0940	701.931	701.940*
4	C1	0945	1,080.142	1,080.155*
5	C1	1025	1,080.141	(1,080.155)
6	A1	1035	966.724	966.736
7	A2	1045	922.873	922.884
8	C1	1050	1,080.141	(1,080.154)
9	A3	1100	881.068	881.078
10	A4	1115	841.599	841.609
11	C1	1120	1,080.142	(1,080.154)
12	A5	1130	804.828	804.837
13	A6	1145	771.115	771.124
14	C1	1150	1,080.142	(1,080.154)
15	C1	1300	1,080.138	(1,080.154)
16	T1	1305	862.473	862.486
17	T2	1315	825.111	825.125
18	C1	1320	1,080.135	(1,080.154)
#	**C1 To**	**Time**	**Observed Distance D_{obs} (meters)**	**Corrected Distance (meters)**
19	C2	1400	566.136	566.144*
20	C3	1405	1,080.133	1,080.149*
21	C4	1410	984.112	984.128*
22	C2	1415	566.139	566.147*
#	**C4 To**	**Time**	**Observed Distance D_{obs} (meters)**	**Corrected Distance (meters)**
23	C1	0835	984.140	984.137*
24	C2	0840	1,133.034	1,133.030*
25	C3	0845	943.206	943.203*
26	C1	0850	984.139	984.136*
27	C1	0900	984.138	(984.134)
28	A1	0905	1,025.543	1,025.540
29	A2	0915	1,036.993	1,036.992
30	A3	0925	1,051.857	1,051.858
31	C1	0930	984.132	(984.134)
32	A4	0940	1,069.987	1,069.991
33	A5	0945	1,091.232	1,091.238
34	A6	0955	1,115.403	1,114.411
35	C1	1000	984.126	(984.134)
36	T1	1010	962.289	962.297
37	T2	1020	968.747	968.756
38	C1	1025	984.125	(984.134)

(Continued)

#	C2 To	Time	Observed Distance D_{obs} (meters)	Corrected Distance (meters)
39	C1	1230	566.149	566.147*
40	C4	1235	1,133.149	1,133.027*
41	C3	1240	701.942	701.940*
42	C1	1245	566.149	(566.146)
43	A1	1250	398.112	398.110
44	A2	1300	337.318	337.316
45	A3	1310	276.622	276.621
46	C1	1315	566.149	(566.146)
47	A4	1320	216.041	216.040
48	A5	1330	155.797	155.796
49	A6	1335	96.408	96.408
50	C1	1345	566.148	(566.146)

Note: * - Denotes length corrected from temperature and pressure measurements. ( ) - Denotes true length.

(8) When the lines have been reduced to the spheroid, the next step is to define the size and shape of the control figure, in this case a doubly braced quadrilateral. There are several ways to do this. One way is, since the figure contains four triangles, these may be individually treated in the same manner as the triangle in Figure 11-6a.

Another way would be to use the means of the six lines in the figure and adjust these by means of a quadrilateral adjustment. This is the technique that was used in the present case to obtain the following adjusted lengths:

C1 to C2	566.146 meters
C1 to C3	1,080.154
C1 to C4	984.134
C2 to C3	701.940
C2 to C4	1,133.029
C3 to C4	943.202

(9) The control figure may be fit into an existing coordinate system or a local system may be devised just for the dam. For the fictitious dam, a local system was used. C4 was selected as a starting point and was assigned coordinates of x = 1,000.000 and y = 1,000.000. The coordinates of C3 were then chosen to place C3 at a distance of 943.202 m from C4; they are x = 1,943.202 and y = 1,000.000; The placement of C4 and C3 has determined the scale and orientation of the figure. Using the positions of C3 and C4 and the appropriate lengths, the positions of C1 and C2 can be determined to be:

C1: x = 1,366.527
y = 1,913.333

C2: x = 1890.936
y = 1699.991

(10) **NOTE: The establishment of the control figure needs be done only once. From that time on, it is only necessary to check for movements of the control monuments. This may be done by comparing observed ratios taken at some later time with the original set.**

(11) Returning to Table 11-7, one may now calculate the corrected lengths D_{corr} to the stations on the top and toe of the dam from the control monuments. This is done by using reference lines to make refractive index corrections.

(12) Measurements 15 through 18 from Table 11-7 are given in Table 11-8.

Table 11-8
Changes of Correction Factor with Time

#	C3 To	Time	D_{obs} (meters)	Correction Factor	D_{corr}* (meters)
15	C1	1300	1,080.138	1.0000148	(1,080.154)
16	T1	1305	862.473	1.0000155	862.486
17	T2	1315	825.111	1.0000169	825.125
18	C2	1320	1,080.135	1.0000135	(1,080.154)

* () denotes true length.

(13) At 1300, when the distance to C1 was measured, the observed distance, D_{obs}, was found to be 1,080.138 m. This line, C3 to C1, is a part of the control figure, and its correct length has been determined to be 1,080.154 m. The atmospheric correction at 1300 may then be found by dividing. The correction is 1,080.154/ 1,080.138 = 1.0000148. Later, at 1320, the atmospheric correction has become 1.0000176. Assuming the change in correction has been linear as a function of time over the 20-minute interval, we may calculate the correction factor at 1305 and 1315 when observed distances were measured to T1 and T2. Multiplying the observed distance by the corresponding atmospheric correction gives the corrected distance, D_C, to T1 and T2. Thus in Table 11-7, the values in parenthesis in column 5 are the correct or true lengths of reference lines, and the values without an asterisk or parenthesis are the corrected lengths that have been calculated from reference lines.

(14) Finally, with the corrected lengths and the coordinates of the control monuments from which they were measured, it is possible to calculate the positions of the points on the dam. Because three lengths were measured to stations on the crest of the dam, three solutions will be obtained. Geometrically, some solutions will be superior to others. For stations at the toe of the dam, only one solution is possible.

(15) In Table 11-9, positions of the crest and toe markers are given for various line combinations, In the case of the crest markers, an adjusted position is also given.

(16) If desired, alignment may be determined from positions. Using the crest stations A1 and A6 as end points, the alignment of A2 through A5 is given in Table 11-10. T1 and T2 are also included in the alignment to help monitor any tilt in the dam. Alignment done from positions is not affected by curved dams, by bends, or by differences in elevations.

Table 11-9
Crest and Toe Station Positions

Station	X	Y	From
A1	1,533.713	1,875.726	C2 to C3
	1,533.710	1,875.720	C2 to C4
	1,533.705	1,875.723	C3 to C4
	1,533.709	1,875.722	Adjusted
A2	1,590.161	1,852.688	C2 to C3
	1,590.158	1,852.682	C2 to C4
	1,590.153	1,852.685	C3 to C4
	1,590.157	1,852.684	Adjusted
A3	1,646.583	1,829.648	C2 to C3
	1,646.588	1,829.656	C2 to C4
	1,646.594	1,829.652	C3 to C4
	1,646.589	1,829.653	Adjusted
A4	1,703.041	1,806.615	C2 to C3
	1,703.038	1,806.609	C2 to C4
	1,703.033	1,806.613	C3 to C4
	1,703.037	1,806.612	Adjusted
A5	1,759.465	1,783.585	C2 to C3
	1,759.467	1,783.588	C2 to C4
	1,759.470	1,783.584	C3 to C4
	1,759.468	1,783.586	Adjusted
A6	1,815.919	1,760.547	C2 to C3
	1,815.915	1,760.542	C2 to C4
	1,815.912	1,760.545	C3 to C4
	1,815.915	1,760.544	Adjusted
T1	1,568.152	1,776.672	C3 to C4
T2	1,608.187	1,754.053	C3 to C4

Table 11-10
Alignment

Station	Distance from A1 (meters)	Distance off Line (meters)*
A2	60.968	0.00
A3	121.919	- 0.001
A4	182.888	+ 0.001
A5	243.836	- 0.004
T1		+78.691
T2		+84.505

* + = Downstream.
- = Upstream.

Chapter 12
Periodic Deflection and Settlement Measurement Surveys (PICES)

12-1. General

This chapter presents guidance for performing field measurements used to determine horizontal deflections and vertical settlements/displacements in structures monitored under the USACE PICES program. This material is intended to supplement and complement EM 1110-2-4300 and should be used in conjunction with the guidance in that reference. Portions of this guidance is in the form of contract specifications, as are used for contracting some aspects of PICES surveys. Standards and specifications for performing precise vertical settlement measurements, crack/joint measurements, and micrometer alignment surveys are covered. These standards are taken from specifications developed by the Jacksonville District in the early 1980s. They were used for both in-house and contracted PICES surveys, and were attached to contract scopes of work. They are generally representative of most USACE deformation monitoring requirements; however, they may not be applicable for PICES surveys on all USACE structures.

12-2. PICES Settlement Monitoring Surveys - General

This section covers standards and specifications for performing precise differential leveling surveys, as required to monitor settlements in concrete and embankment structures. The standards described are developed around precision leveling instruments used for long-distance geodetic leveling runs -- typically compensator (self-leveling) instruments with parallel plate micrometers and dual-scale invar rods with supporting struts. For many structures where level runs are relatively short, this high precision equipment and procedures represents “overkill”-- adequate results may be obtained with more traditional leveling methods (e.g., three-wire or even single-wire observations). Recent improvements in electronic total stations and development of bar code leveling methods are expected to radically change precise vertical survey techniques.

12-3. Vertical Settlement Measurements Using Precise Leveling Techniques

a. Vertical settlement is determined by precision differential leveling methods performed using compensatory auto-collimation leveling instruments with fixed or attached parallel plate micrometers, observing invar double (offset) scale metric rods with supporting struts. In general, one to three fixed reference points (bedrock benchmarks) are used to check for potential movement of various points on the structure. One of the reference points (situated well away from the structure) is held fixed with all subsequent PICES vertical changes tabulated relative to this fixed reference point. Normally an arbitrary elevation is assumed (i.e., 100.000 m); thus the entire structure is on an arbitrary reference datum. Vertical ties between reference bedrock benchmarks are performed only to monitor potential movement on the reference points -- and to enable selection of the best reference point to hold fixed when two or more deep bedrock benchmarks (DBMs) are available. PICES settlement surveys are made infrequently, normally at 6- to 18-month intervals, or sometimes not at all unless there is a suspicion of structural distress.

b. PICES leveling shall be performed in conformance with the methods and accuracy specifications contained in NOAA Manual NOS NGS 3, Geodetic Leveling. Minor variations therefrom are contained within these specifications. Those performing PICES survey work are expected to be thoroughly familiar with the contents of this reference manual. Other applicable references include: ER 1110-2-1806, EM 1110-2-1911, EM 1110-2-2300, and EM 1110-1-1904. This last reference provides guidelines for calculations of vertical displacements and settlement of soil under shallow foundations supporting various types of structures and under embankments.

12-4. Required/Expected Accuracy of Vertical Measurements Using Differential Leveling

Settlement measurement accuracies, relative to the presumed rigid deep bedrock benchmarks, should be on the order of +/-0.002 m, depending upon the leveling distances involved, stability of the reference benchmark, short-term vertical movement in the structure, and other factors. The differential leveling procedures employed should yield 0.001 m (short term) accuracies for most typical structures. High precision geodetic leveling procedures have been modified to meet specific PICES accuracy requirements.

12-5. Definitions

The following definitions apply to differential leveling monuments, instruments, and procedures:

Structure Settlement Point: any point on a structure to which differential leveling is run. May be a disc (Benchmark or BM), survey marker disc, grouted bronze plug with insertable caps, etc.

DBM: Deep (bedrock) Benchmark.

RDBM: Reference Deep (bedrock) Benchmark. This is the same as a DBM except that this mark is that held fixed for relative movement analysis.

Double Run/Double Rod (DR/DR) Leveling: for new PICES projects or for re-observing misclosures from single run/double rod leveling.

Single Run/Double Rod (SR/DR) Leveling: for recurring PICES projects, i.e., one-way leveling. Primary leveling procedure employed for most projects.

Spur Leveling: one-way single rod leveling used for tying in points in a close proximity. Spurred from main DBM or RDBM Single Run/Double Rod level line.

Internal Misclosure: results from Double Run/Double Rod leveling (forward/backward runs).

External Misclosure: comparing single run or spur leveling results with data obtained on previous PICES observations.

12-6. PICES Project Requirements and Instructions for Settlement Observations

Project instructions for continuing PICES projects will generally list (in tabular form on previous PICES reports) those structural settlement points requiring updated elevations (or differential changes from previous readings), and the specific RDBM to be held fixed. Specific leveling routes will be at the discretion of the field party chief, as is the need for single run or double run leveling. Construction requirements for either new deep bedrock benchmarks or new structural settlement monitoring points will be detailed if required.

12-7. Instrumentation and Equipment Requirements

The instrumentation used should meet the requirements for First-Order geodetic leveling -- employing either spirit levels or compensator levels with micrometers, or bar code levels.

a. Recording formats. Any acceptable version of the NGS Micrometer Leveling form may be used -- an 8.5-inch by 11-inch loose-leaf format. Field books and data recorders are also acceptable. Level sketches and abstracts shall also be prepared on 8.5- by 11-inch size loose-leaf paper. A sample recording form is shown in Figure 12-1.

b. Observing procedures. Precise differential leveling methods shall conform to the general methods outlined in Chapter 3 of NOAA Manual NOS NGS 3, as modified herein for either double rod, single rod, or spur leveling. All level lines between RDBMs, DBMs, and structure monitoring BMs shall be run using SR/DR precise leveling methods. DR/DR leveling methods are required only on new PICES projects or when single run lines do not meet external misclosure tolerances.

c. Single or Double Run/Double Rod leveling procedures.

(1) Sections shall not exceed 1 km in length.

(2) Each section shall start and end with the head rod (Rod A) on the BM or reference point. This prevents an accumulation of rod index errors due to an uneven number of setups.

(3) The head rod (Rod A) is always observed first on each setup, whether backsight or foresight.

(4) The instrument is leveled with the telescope pointing toward the head rod (Rod A) -- thus alternating toward the backsight and foresight at alternate instrument stations.

(5) The line of sight between instrument and rod should always be higher than 0.5 m above the ground surface. Maximum line of sight distance shall not be greater than 40 m.

(6) Any given setup will be re-observed if the disagreement between the left and right side scale elevations on either rod exceeds 0.25 mm for that setup.

(7) Backward and forward stadia distances can differ by no more than 2 m per setup and 4 m accumulated along a section. (Note: 1-m tolerance for spur leveling.)

(8) Turning plates should not be used on turf; driven turning pins will be required in this type of terrain.

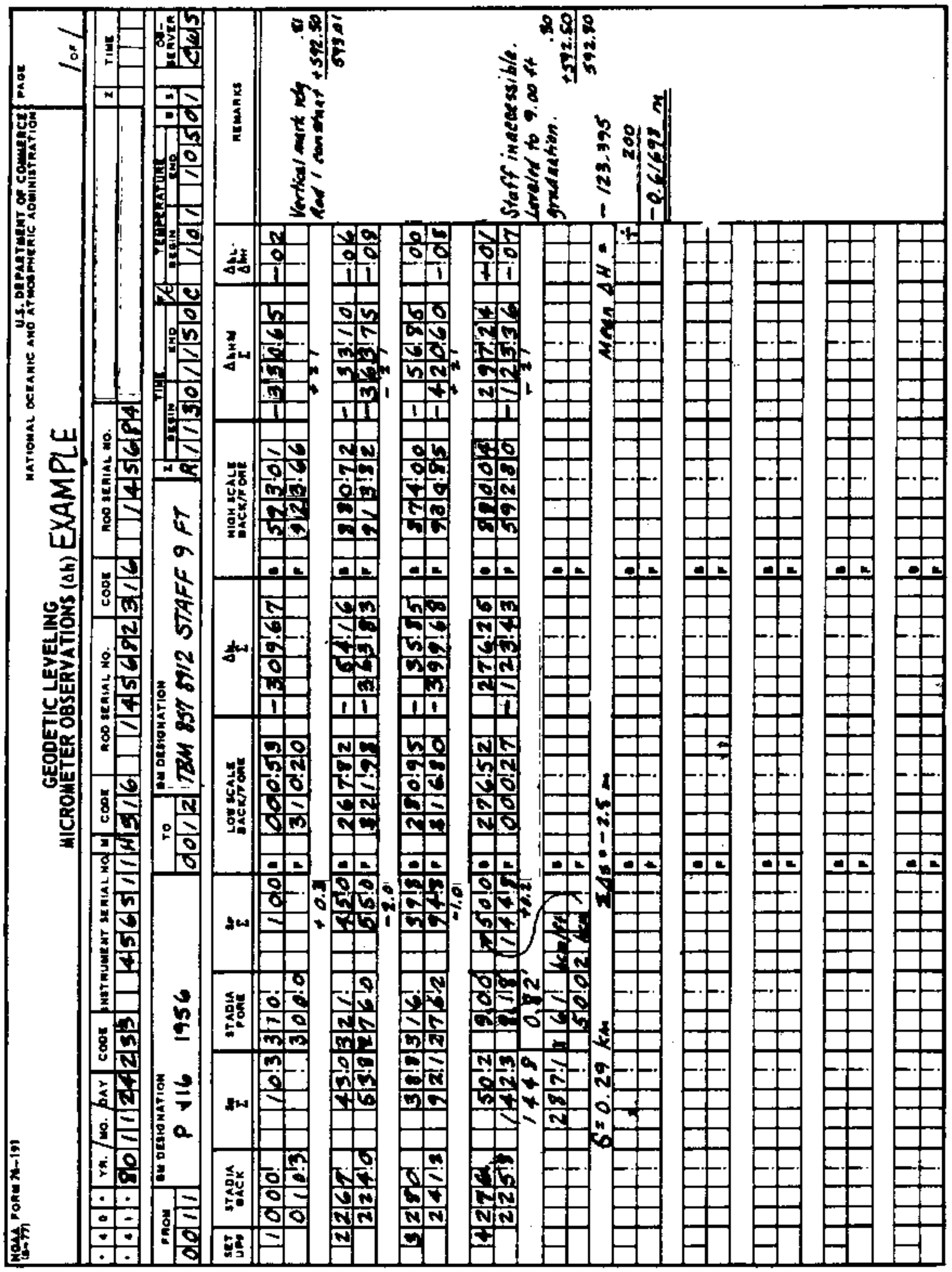


Figure 12-1. Micrometer observations

Turning plates should only be used on pavement or hard packed soil and only when absolutely necessary.

d. General connections between RDBM and DBM for existing PICES projects.

(1) Use SR/DR leveling via shortest route between RDBMs and DBMs.

(2) Double run only if external misclosures exceed tolerances.

(3) Ensure direct route from RDBM to structure monitoring points.

e. Spur leveling techniques or open-ended lines.

(1) Use single rod only (Rod A).

(2) Up to three instrument setups are allowable. (If more than three, double run back to starting BM and verify internal misclosure.)

(3) Start spur from a rigid BM and not from a TBM.

(4) Keep backsight/foresight distances within 1 m -- individual and accumulated.

(5) Multiple foresight shots are allowable from a single backsight assuming distances are allowable. Record similar to conventional leveling.

(6) Verify external misclosure on site -- re-observe, as required, using DR/DR methods.

(7) Single rod leveling methods may be employed for main connection lines between DBMs and BMs, should such methods prove to be more efficient.

(8) Observing and recording methods are similar to conventional leveling procedures, as modified for micrometer leveling.

f. Internal misclosure tolerances. Newly established points or for re-observations when external misclosures reject on single runs.

MISCLOSURE TOLERANCE = M = $+3 \text{ mm} \sqrt{K}$

where K is measured in kilometers

If K < 1,000 m, then M = $+3 \text{ mm} \times \sqrt{K}$

MINIMUM M: 1 mm @ K less than 0.33 km

Rerun if in excess.

g. Instrument calibration requirements.

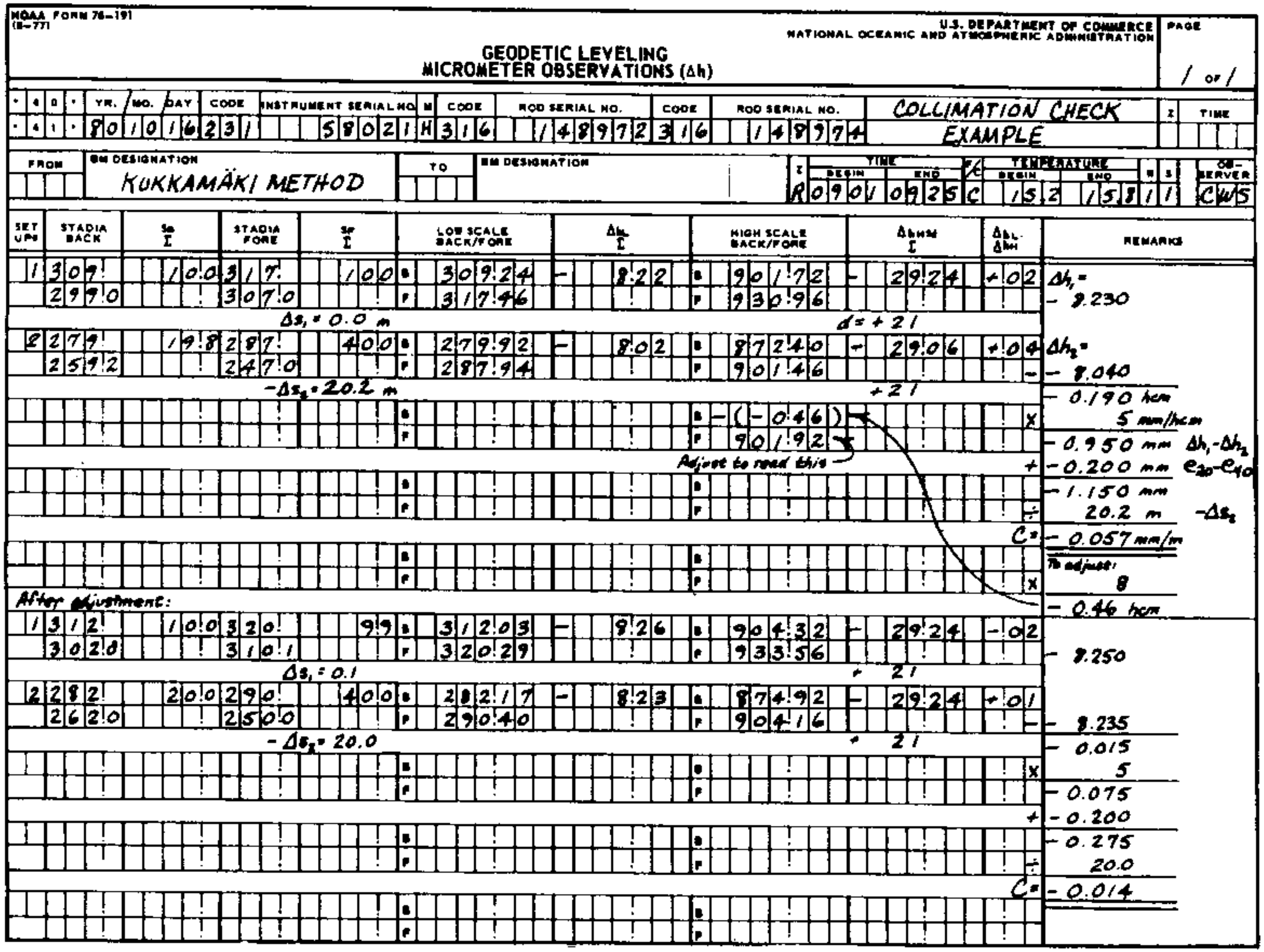
(1) Precise level rods and the instrument will be lab calibrated/maintained at least annually.

(2) C-factor collimation calibration. The C-factor shall be determined at the beginning of each PICES structure observation in accordance with the procedures outlined in Section 3 of NOAA Manual NOS NGS 3. The C-factor determination is done according to Kukkamaki's method and is also referred to as the Peg Test. The C-factor shall conform to the reject/readjustment criteria of Table 3-1 of NOAA Manual NOS NGS 3 which is 0.005 cm/m. Daily C-factor calibrations are not essential provided the instrument is consistently falling within 0.004 cm/m and backsight/foresight distances (individual setup and accumulated) stay within 1 m/2 m, respectively. In any event, C-factor calibrations shall be performed at least twice weekly when performing continuous leveling at a single PICES structure; upon commencing leveling at a new structure; or daily if the C-factors exceed prescribed limits. (See Figure 12-2.)

12-8. Leveling Computations and Reductions

a. Micrometer leveling data sheets will be checked in the field by an independent person (initial sheet accordingly), with the resultant differential elevation (DE) for each run clearly noted, along with pertinent plug offset characteristics, if any, and accumulated stadia lengths per circuit/section. Misclosure tolerances will thus, by this manner, be field verified/confirmed; accordingly, there shall be no further office verification of the basic observational data sheets -- the validity of internal/external closures having been made in the field.

b. Field sketches of level circuits/section/loops/spurs shall clearly show observed elevation differences, leveling direction, and stadia distances, all taken directly from the (checked) micrometer leveling recording forms. From such a sketch, elevations may be easily carried forward from the RDBM -- an essential computation in verifying external misclosures and should be stapled to all the data sheets acquired for an individual PICES structure. Elevations carried forward (from the RDBM) may be listed on a separate sheet, (i.e., an abstract).



NOAA FORM 76-191 (8-77)

U.S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION

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GEODETIC LEVELING
MICROMETER OBSERVATIONS (Δh)

• 4 0 •	YR.	MO.	DAY	CODE	INSTRUMENT SERIAL NO.	M	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	TIME
• 4 1 •	90	10	16	231	5802	1 H	316	148972	316	148974	COLLIMATION CHECK EXAMPLE		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
	KUKKAMÄKI METHOD			R	0901	0925	C	152	158	1	1	CWS

SET UPS	STADIA BACK	Sb Σ	STADIA FORE	Sf Σ		LOW SCALE BACK/FORE	Δh Σ		HIGH SCALE BACK/FORE	Δhhs Σ	Δhl-Δhh	REMARKS
1	3094	1003	317	100	B	30924	- 822	B	90172	- 2924	+02	Δh_1 =
	2990		3070		F	31746		F	93096			- 8.230
		Δs_1 = 0.0 m							d = + 21			
2	2279	198	287	400	B	27992	- 802	B	87240	- 2906	+04	Δh_2 =
	2592		2470		F	28794		F	90146		-	- 8.040
		$-\Delta s_2$ = 20.2 m							+ 21			- 0.190 hcm
					B			B	-(-046)		x	5 mm/hcm
					F			F	90192			- 0.950 mm $\Delta h_1 - \Delta h_2$
									Adjust to read this		+	- 0.200 mm $e_{20} - e_{40}$
					B			B				- 1.150 mm
					F			F			÷	20.2 m $-\Delta s_2$
											C =	- 0.057 mm/m
					B			B				To adjust:
					F			F			x	8
After adjustment:												- 0.46 hcm
1	3121	1003	320	99	B	31203	- 926	B	90432	- 2924	-02	
	3028		3101		F	32029		F	93356			- 8.250
		Δs_1 = 0.1							+ 21			
2	2282	2002	290	400	B	28217	- 823	B	87492	- 2924	+01	
	2620		2500		F	29040		F	90416		-	- 8.235
		$-\Delta s_2$ = 20.0							+ 21			- 0.015
					B			B			x	5
					F			F				- 0.075
											+	- 0.200
					B			B				- 0.275
					F			F			÷	20.0
											C =	- 0.014
					B			B				
					F			F				

Figure 12-2. C-factor collimation calibration, Kukkamaki example

12-9. Office Computations, Reductions, and Adjustments

Presuming the above field computations, checks, and sketches are adequately performed, no formal office reverification of field data need be performed. Only a verification of the abstracted elevations should be required -- and compiling these values into the current PICES report. Examples of settlement tabulations and graphical plots showing historical trends are found in EM 1110-2-4300.

a. In general, no rigorous least squares (condition) type adjustments are necessary for PICES projects of limited magnitude wherein no sophisticated weighting method is effective. Redundant elevations (i.e., computed from different loops on circuits from the RDBM) may be simply averaged (meaned) regardless of lengths run. Since most PICES structures will involve single run lines run directly from the RDBM, final adjusted structure elevations are simply algebraic accumulations of DEs from the RDBM -- using field verified sketch/abstract data.

b. More sophisticated leveling adjustment procedures may be necessary in the case of:

- Newly established projects.
- Settlement anomalies (DBM or structure points).
- Abnormal movement of RDBMs or DBMs.
- Redefinition of RDBM using past data.

c. Tabulate carried forward elevations or averaged elevations, from the field sketch/abstract, holding the historical RDBM fixed and computing changes in elevation from prior PICES observations. Anomalies should be noted on the PICES report tabulation. Recommendations to change the RDBM (to another DBM) should be noted and pursued accordingly.

d. Reported elections of screw-in (or male insert) structural reference points (i.e., females) should clearly identify the type of male insert employed on the PICES report for the structure. Such identification (by male part/serial number) is critical if male insert elevations are eccentric to the female plug. Monitoring of such types of points should be avoided in the future -- in lieu of standard bronze discs where no vertical eccentricity problems exist. (Loss of a male insert nullifies all future PICES settlement comparisons.) Pending eventual conversion to standard discs, recordation of male inserts is critical.

e. Record elevations to the nearest ten-thousandth (0.0001) m on final reported elevations and settlement changes. Presumptions of higher short-term or long-term accuracies are unwarranted, given procedural methods, misclosure tolerances, and potential settlement of DBMs and structures. Elevations (and elevation differences) on field sketches and abstracts should be tabulated to the nearest 0.0001 m.

12-10. PICES Micrometer Alignment Deflection Measurements-General

This section describes the techniques and specifications for measuring relative deflections using theodolites and precision micrometer caliper targets. Micrometer deflection measurements will yield accuracies that are far superior to geodetic techniques described in previous chapters; however, these deflection measurements are relative and not necessarily absolute movements. Accordingly, micrometer measurements are intended to monitor relatively short-term deflections in a structure monolith, wall, tower, etc., due to varying hydraulic head, temperature, curing, or other physical effect. Forced-centering of both the instrument and micrometer target is critical if accurate repeatability is desired. Supplemental information on micrometer deformation observations is found in EM 1110-2-4300.

12-11. Micrometer Deflection Observations

Short-term horizontal deflections or deformations of points on structural sections can be easily monitored by observing alignment variations with a precision theodolite relative to (presumed) fixed points on a baseline not influenced by the structure. In essence, the deflection of a point relative to a fixed baseline is observed either by micrometer target methods or by observing the actual deflection angle directly (and computing the relative lateral movement indirectly). The theodolite and reference target(s) must be set up on concrete instrument stands using rigid forced-centering devices. Structure monitoring target points (or plugs) are normally set (grouted) within ±0.5 inch from the reference baseline from which deflections are referred.

12-12. References

Added background on relative deflection measurement techniques are described in EM 1110-2-4300.

12-13. Expected Deflection Observation Accuracies

Given the centering limitations of a precision theodolite, cross hair delineations, theodolite vertical alignment limitations, and numerous other factors, absolute horizontal deflections can be measured to approximately ±0.01 inch over typical distances involved. This value is relative to the fixed alignment baseline on which the theodolite and reference target are set. In some instances (e.g., along lock walls) this fixed baseline may not be perfectly stable, and this may further degrade the ultimate accuracy of measuring deflections in individual structural sections. In addition, other short-term effects may mask the relative accuracy of deflection observations. These include short-term temperature changes (sun-cloud), power plant machinery, lock operation, etc. Observers should exercise judgment regarding the ultimate accuracy of micrometer measurements when local conditions are impacting observations.

12-14. PICES Micrometer Alignment Requirements and Instructions

PICES alignment requirements for each structure will be listed in tabular form on project instructions -- identifying the baseline reference points (instrument/target stands), the deflection points to be observed, and structure loading requirements (e.g., lock fill elevations). Requirements for establishing new alignment points and constructing reference baseline instrument/target stands will be detailed as required.

12-15. Instrumentation Requirements

a. Wild T-2 or T-3 theodolite with forced-centering tribrach. Other similar instruments (e.g., total stations) may be used.

b. Targets: Inverted “V” or conic plug inserts.

c. Alignment micrometer with forced-centering plug insert. Inverted “V” or conic targets mounted on micrometer. See EM 1110-2-4300 for details on forced-centering monument construction and monolith alignment marker design.

d. Recording format. Use a standard field survey book for both observations, computations, and adjustments of data.

12-16. Observing Procedures

a. Theodolite and target are force-center mounted on each end of the reference baseline. Baselines typically range from 100 to 1,000 feet in length, depending on the structure. The baseline is established perpendicular to the direction in which deflection observations are required.

b. After force-centering the theodolite, accurately level theodolite to its reversing point -- relevel to the reversing point before each observation. This theodolite leveling/releveling procedure is critical. In addition, parallax is removed from the theodolite’s cross hairs. The reference target on the opposite end of the reference line is aligned by forced-centering, and ensuring the target is aligned vertically over the plug center.

c. The theodolite’s vertical hair is centered on the reference target and five (5) alignment deflections are observed with the theodolite in Direct position only. For each observation, initial on the reference target, drop vertical line of sight, and move the alignment micrometer/target to collimation with T-2. Radios may be required for communication between the instrument man and micrometer operator. Ensure micrometer device is as level as possible. Also record pertinent structural load and temperature conditions.

d. Observe five deflections with the alignment micrometer in the LEFT position (i.e., micrometer is to left of baseline as viewed from the theodolite’s position).

e. Rotate alignment micrometer 180 degrees to its RIGHT position, and observe five deflections (theodolite is still in Direct position). Relevel theodolite (to its reversing point) before sighting on the reference target. (Leveling of the micrometer is critical on the lateral axis.)

f. Always run the micrometer against the spring. After each deflection observation, the micrometer should be backed off a few hundredths of an inch.

g. Read alignment micrometer to nearest ±0.001 (thousandth) inch.

h. Determine the mean of both LEFT and RIGHT micrometer observations (five each) to nearest 0.001 inch. The difference between the mean of the LEFT set and the mean of the RIGHT should not exceed ±0.02 (two-hundredths) inch. If the difference between the two means does exceed this limit then both the LEFT and RIGHT set must be reobserved. Large variations probably indicate poor target centering, parallax not eliminated, or mislevelment of the theodolite.

i. Check internal rejection criteria. Reject if measurement is $\pm$0.02 inch from the mean.

j. Check external rejection criteria. Check previous PICES report for potential blunders.

k. There are no micrometer calibration requirements during observations since reversing the micrometer to the LEFT and RIGHT positions eliminates index error in the device.

l. There is no need to reverse the theodolite, given the small vertical deflections involved, and higher errors inherent from other sources. If significant vertical deflections are involved, then non-verticality of the theodolite could inject errors. Also, the vertical cross hair should be periodically checked and adjusted, in accordance with the manufacturer's manual. The theodolite itself should be serviced and calibrated annually.

12-17. Field Computations and Reductions

a. The final left/right deflection angle will be computed in the field after each alignment observation, and noted in the field survey book. Accordingly, no office recomputations will be required. All field reductions shall be independently checked in the field.

b. From the meaned micrometer LEFT/RIGHT readings, compute the adjusted deflection as follows:

Deflection = D = (ML - MR)/Z

where

D = "+" RIGHT deflection off baseline to structure point as viewed from theodolite position

ML = mean of five LEFT micrometer readings on baseline

MR = mean of five RIGHT micrometer readings on baseline

c. Round adjusted deflection to the nearest 0.01 inch.

d. The sum of the micrometer LEFT and RIGHT means will not necessarily total to 1.000 inch, given the micrometer index errors.

e. Office tabulations. Simply tabulate field-computed deflection values onto the PICES report. Note all local conditions (i.e., water level, temperature, etc.) for each observed deflection. See sample recording formats in EM 1110-2-4300.

12-18. PICES Crack and Joint Measurement Procedure - General

This section describes absolute micrometer joint or crack measurement procedures using micrometers. Relative displacement techniques are not covered -- see EM 1110-2-4300. PICES crack/joint observations are measured relative to grouted bronze plugs set 12 inches (±) on center across a concrete crack or structural construction joint where periodic monitoring is required. Monitoring points are usually set on each adjacent monolith. Monitoring is performed periodically for long-term trends or during short-term load deformation studies. Often, three plugs are set across each crack or joint in a triangular pattern. In most cases, two opposite plugs set perpendicular to the joint/crack plane will be adequate. Expected short-term accuracy is on the order of ±0.0005 inch, relative to the fixed calibration reference bar. Errors due to the nonalignment (vertical) of the crack plugs relative to one another could affect observational accuracy (and long-term repeatability) upwards of ±0.01 inch. Given all of the above errors and uncertainties, estimated long-term crack measurement accuracy is at the ±0.005- to 0.010-inch level, totally independent of short-term movements in the structure due to load or temperature influences.

12-19. PICES Requirements and Instructions

Crack and joint measurement requirements are typically listed in tabular form, including instructions for varying hydraulic head levels against the monoliths, if applicable. Requirements and instructions for setting new monitoring points will be provided as required. Structure loading requirements will also be provided for each new observation point specified.

12-20. Instrument and Equipment

a. Inside micrometer. Any standard machine tooling inside micrometer may be used for crack measurements. Precision calipers may also be employed in lieu of an inside micrometer.

b. Inside micrometer calibration bar. A 12-inch center to center standard reference may be used for all micrometer observations. An independent recalibration of this bar is necessary to monitor long-term stability.

c. Plug inserts. Stainless steel threaded half-inch inserts are used and inserted into the dual or triad points across monolith joints or cracks. Inserts are stamped to ensure consistent use on periodic measurements. The 0.500-inch O/D inserts should be precision machined to an accuracy of ±0.001 inch and verified by micrometer measurement.

12-21. Crack Measurement Techniques

a. Insert plug pins and measure crack or joint distance using an inside micrometer or caliper.

b. Read micrometer/caliper values to nearest 0.001 inch.

c. Read in both directions (i.e., reverse micrometer ends) between crack plugs and mean result to nearest 0.001 inch.

d. Hold micrometer ends as low as possible on each plug pin. Gently rotate each end for minimum distance observation.

e. Readings in each direction should not vary by more than ±0.001 inch unless it can be verified that the crack plugs are grossly misaligned vertically. This can be verified by raising the micrometer at both ends to confirm nonverticality of the grouted plugs. Do not attempt to interpolate between 0.001-inch values. Record a single minimum reading for each direction and mean as required.

f. The following applies to an inside micrometer with dial: Lock micrometer to nearest 0.025-inch division and use dial indicator to obtain minimum distance. (Maximum reading on scale which is subtracted from the preset micrometer value):

Example:

Micrometer set at:	11.475 inches
Maximum dial scale reading (minimum distance):	-0.021 inch
Observed uncorrected micrometer length:	11.454 inches

g. Ensure dial range is within 0.025-inch micrometer setting range to avoid misreadings and ensure relatively constant spring tension.

12-22. Micrometer Calibration Bars

a. The calibration bar is used to ensure the micrometer is accurate by verifying a 12.000-inch center to center distance. The calibration bar should be kept shaded. Calibrate caliper and/or micrometer prior to PICES structure observation, using an independent reference. Single meaned forward/backward micrometer positions on the calibration bar should be observed/recorded to the nearest 0.001 inch.

b. Example of calibration for Starrett Micrometer (similar operation required for Helios Caliper).

	FORWARD	BACKWARD
Micrometer	11.475	11.475
Dial	-0.021	-0.020
Reading	11.454	11.455

Meaned calibration reading = 11.454 inches

CALIBRATION CORRECTION	
(Nominal calibration bar length)	12.000
- (Calibration reading)	-11.454
Calibration correction	00.546 inch

c. The above micrometer and caliper calibration correction is applied to all subsequent crack readings.

d. Typical crack observation (Starrett Inside Micrometer). Typical field book entry:

Cross Florida Barge Canal
Inglis Lock & Spillway

Points:		IL19N4 to IL19N5
19 July 1984		0845 am
Mic	-	Bergen
Notes	-	Noles, Bergen
T	-	86° F, Rain
Lock full @ 36.0’ elev.		

FWD		BACK	
11.475	Mic	11.475	
-0.019	Dial	-0.020	
11.456		11.455	Mean = 11.456 inch

Calibration correction = +0.546 inch

Corrected plug reading (IL19N4 to IL19N5) = 12.002 inches

The corrected plug-to-plug reading (12.002) may be directly inserted on PICES tabulation report; no further adjustments are required.

e. Typical plug pin convention for Triad Crack/Plug Configurations. Three marked pins shall be used in the same plug upon each PICES revisit, per the following convention:

"L"	Lowest numbered crack plug
"H"	Highest numbered crack plug
"b"	"Blank," in medium numbered crack plug

i.e. Inglis Lock

PLUG	PLUG
IL19N4	"L"
IL19N5	Blank
IL19N6	"H"

f. Normally, only one forward/reverse observation will be required for each pair of plugs, approximately a 1- to 2-minute procedure. Additional observations under different structural loading conditions or temperature conditions will not be performed unless specifically requested in the project instructions. In cases where observations are taken over varying points in time or condition, they will not be meaned, given the external structural variability of the measurements. (All PICES observations are only valid for the finite period in which they are taken given the external influences, primarily by the structure and to a lesser extent the instrumentation employed. This applies to vertical, horizontal, and alignment observations also).

g. Internal rejection criteria are ±0.001 inch between each direction reversal and ±0.001 inch from nominal calibration bar constant. Failure to obtain agreement in each direction may be due to nonverticality of the plugs, in which case no re-observations are necessary.

h. There are no external rejection criteria other than a blunder check using prior PICES observations (reports).

i. Standard field survey books for both observations and corrected/adjusted lengths are normally used for record keeping.

12-23. Periodic Calibration Requirements on Micrometer

a. Independent annual calibrations should be performed on the following components:

- Inside micrometer or calipers.
- Reference calibration bar.
- Threaded 1/2-inch plug inserts.

b. Calibrations should be checked over the normal temperature range which these devices are subject to in order to determine if expansion (temperature dependent) corrections become significant.

c. There is no method for eliminating the error due to nonverticality of the plugs other than using identical inserts on each visit. Use of inside/outside precision calipers will eliminate most independent calibration requirements other than the calipers themselves and ensure true roundness and alignment of the threaded plug inserts. The need for a reference calibration bar may also be eliminated.

12-24. Data Computations and Reductions

a. All PICES observations and reductions shall be performed and verified in the field, directly into the field survey book. Micrometer data are corrected for calibration constants as shown above. Quick comparisons should be made with previous PICES observations to preclude against blunders.

b. Office processes are not required other than tabulating field reduced distance onto PICES reports and computing changes from past readings. Standard forms for periodic crack measurements are also found in EM 1110-2-4300.

12-25. PICES: Horizontal EDM Observations-General

The following procedures apply to structural deformation observations utilizing EDMs. EDMs include either standalone or electronic total stations. Distances to structural monitoring points are observed from one or more rigid instrument stands or tripods over "fixed" points. The positioning of the fixed points is not influenced by movement of the structure and is presumed to be static. The fixed points constitute a local baseline for each structure. Structural movement is then determined relative to these external points. The external reference points are set such that the relative movement vector being measured is generally perpendicular to the structure's probable plane of failure. The field method for obtaining the EDM measurements for PICES is referred to as the RLR/Robertson Method, as detailed in prior chapters. The RLR method will provide sufficient relative accuracy measurements for most USACE structural monitoring applications. This section describes the general procedures used in measuring actual distances with typical EDM equipment. Establishment of the reference network and periodic measurement procedures are covered. The RLR method requires no EDM calibration or meteorological conditions correction. Rather, the distance between two fixed points (baseline distance) at the structure is measured and compared to the published distance (distance recorded in the PICES tabulation). If there is a difference upon comparison of the two distances, then either change the ppm reading on the instrument to match the published distance or compute the ratio of the measured distance to the published distance. The distances to the structure points are then measured. If measurements are made for more than 30 minutes, the baseline distance must be checked and the above procedure repeated if there is a difference between the measured and published distances. Continue measuring distances to structure points. Distances measured using the option where the ppm is adjusted to match the published distances can be entered directly into the tabulation, and the change and cumulative change computed and also entered into the tabulation.

12-26. Required Accuracy

The estimated accuracy of any individual EDM length observation measured over typical distances is ±0.002 to 0.005 m. This estimate is based on potential error sources involving:

- EDM rated precision point centering limitations -- instrument and reflector.
- Atmospheric corrections and reductions, or variations during RLR method.
- Short-term structural deformations occurring during actual PICES measurement process.
- Numerous other factors well documented in geodetic reference texts and manuals.

Since most PICES projects involve relatively short EDM lengths (i.e., between 100 and 1,000 m), errors due to poorly observed atmospheric corrections and slope/horizontal reductions are relatively insignificant (less than 1-3 mm) compared to EDM instrument limitations and instrument/reflector centering uncertainties, among others. In some instances, over rigid forced-centering bases, accuracies may be obtained down to the 0.003 m level.

12-27. PICES Requirements and Instructions

Requirements for recurring PICES EDM observations will be received in either tabular or plan form, detailing the lengths to be observed to each particular structural monitoring point. A single length implies a unit vector comparison. Requirements for establishing new monitoring points and constructing reference instrument stands will be detailed as necessary.

a. The EDM instrument must be paired with specific (numbered) reflector.

b. Both the instrument and reflector should be mounted using forced-centering devices. Wild tribrachs with standard target level vials may be used to level tribrach housing over plugs. Threaded aluminum rods for direct insert in monitoring plugs may be used to support reflectors.

c. Engineer scales may be used for measuring instrument/reflector heights over base, as needed for slope/horizontal distance reduction.

d. Standard Wild-type tripods with Wild tribrachs should be used for nonforced-centering mounts. Use of tripods is not recommended for PICES work due to difficulty in accurately plumbing instrument/reflector over marks.

e. Standard field survey books may be used for recording all observations, computations, and reductions.

12-28. General EDM Observing Procedures for Lines Less Than 1,000 M in Length

a. Only Wild interchangeable tribrachs shall be used such that the instrument/reflector may be readily interchanged without affecting centering of the tripod/tribrach mount. Any other type of equipment shall be considered unacceptable.

b. Only one instrument/reflector combination shall be used for a particular line. The serial numbers of the instrument and reflector shall be recorded for each observation to ensure and verify this fact.

c. Accurately centering instruments, reflectors, or tribrachs over the marks is one of the most critical processes involved in short-range EDM observations. In order to ensure that centering errors are minimized, the following procedures should be followed for all structural deformation EDM work.

(1) Wild tribrachs shall be accurately plumbed using the built-in optical plummet. Optical plummets shall be calibrated at the beginning of each project using the procedures outlined in the manufacturer's manual. Failure to perform/certify/record this calibration process can be grounds for rejecting all subsequent EDM data obtained with an uncalibrated tribrach.

(2) Tripod heads shall be aligned as nearly horizontal as is possible, prior to final centering procedures.

(3) Final tribrach leveling and centering shall be performed using a level vial from either a mounted theodolite or a standard Wild target. The built-in bull's-eye level is not considered accurate enough for this process and should only be used for rough tripod head alignment. All centering leveling vials should be calibrated at the beginning of each project and this fact so recorded during that process.

(4) In general, all tripod/tribrach centering shall be performed to an accuracy of ±1 mm.

(5) When using a tripod in lieu of forced-centering, and weather conditions permit, optical centering may be checked using a standard plumb bob. Significant disagreement (say more than 1 mm) would be grounds for immediately recalibrating all optical plummets and level vials.

d. Once tripods and tribrachs have been accurately centered over each end of the line, then the EDM instrument and reflector may be inserted into the tribrachs without further adjustment. Extreme care shall be taken to avoid disturbing the tribrach centering during the insertion and measurement process. Since each line may be measured in both directions, the instrument/reflector swapping process shall be carefully executed to avoid disturbance of the centering alignment.

e. Upon completion of all observations from a particular tripod, a final level and centering check shall be performed to ensure no movement has occurred during this process. If no movement is detected during this final check, then the entire observation process shall be repeated.

f. In precise surveys, towers, stands, and tripods must be substantial. The use of driven stakes or some type of quick setting cement or dental plaster for tripod leg support may be required. Catwalks, support away from tripod legs, may be necessary under some soil or platform conditions to ensure that the instrument/reflector is unaffected by motion around it.

g. When relatively short reflector rods are screwed directly into grouted plugs, it is critical that the same rod is used for each successive PICES project. Therefore, the rod number should be recorded such that this rod is always used at a particular plug. Reflector HI's should be kept as low as possible to minimize potential nonverticality of the rods.

12-29. EDM Observing Repetitions

a. Five (5) repeated observations should normally be taken, or approximately 30 seconds of observations. A second series of 5 observations shall be taken after repointing on the target. Consult manufacturer's recommendations for additional guidance.

b. Record repeated observations to the least count on the EDM -- the nearest 0.001 or 0.0001 m. Mean the result to the same degree of precision.

c. Distances need only be observed in one direction when the instrument is set up on positive centered concrete instrument stands. Measure both directions when using tripod supports. On some projects, double measuring need only be performed if a one-way distance deviated over 5 mm from previous PICES observations.

d. Distances between fixed instrument stands, if required, shall be observed in both directions.

e. The following observations shall be made for each EDM distance measured.

- Heights of instrument and reflector (HI/HR) shall be accurately measured and recorded at each end -- to nearest 0.01 foot.

- Instrument stands with elevations determined relative to domed plugs must be corrected accordingly when HI measurements are relative to the plug base.

- The EDM instrument shall have the electro-optical center calibrated and marked such that accurate instrument heights may be determined for each observation. Likewise, the optical center of the reflector shall be equally calibrated and marked.

f. The previously determined system constant for an EDM shall be recorded in the field book for each observation. Accordingly, the instrument and reflector serial numbers must be noted in the field book to ensure the calibrated pair is correct. Incorrect instrument/reflector serial numbers and system constant will result in rejection of all data.

g. The rigid reflector rod number, when used, must be recorded in the field book.

h. Figure 12-3 depicts a typical field EDM observation and reduction.

12-30. Internal EDM Rejection Criteria

a. The spread from the mean of the observations (2 sets of 5) shall not vary by more than 0.002 m. If 0.002 m is exceeded, re-observe the series.

b. Measurements taken in both directions should agree to 0.002 m after measurements are corrected for slope and atmospheric conditions, as required.

12-31. EDM Computations, Reductions and Adjustments

a. In performing the procedures outlined above, final corrected horizontal distances will be computed and verified/checked in the field. The following corrections/reductions will be performed/corrected in the field:

- System constant.

- Horizontal eccentricities (if any).

- Slope to horizontal correction.

b. No corrections to sea level need be applied in PICES projects involving short lines (i.e., less than 1,000 m) or projects near sea level (e.g. in Florida). Final reduced slope distances (corrected for atmospheric observations) shall be reduced to horizontal distances using the elevation differences determined from differential levels. Zenith distances (vertical angles) are rarely used for determining EDM elevations. Accurate heights of instruments are critical; accordingly, these values shall be precisely measured for each observation (using a rod scale or other equally accurate method). Field notes and computation/reduction recording forms shall show the application and/or consideration of all the correction factors described above. Reduced horizontal lengths should be checked and initialed in the field.

c. In some cases where only relative vector movement is being monitored, slope-horizontal reductions need not be applied if distances are relatively long and HIs are of average amounts.

12-32. Office Computations and Adjustments

Since incoming field observations are fully corrected and reduced to horizontal, no further office adjustment of the individual length observation is necessary. PICES report tabulations are performed in the standard manner showing single lengths plus relative changes from past observations.

PROJECT	PAGE	COMPUTED BY	DATE
PICES: HORIZONTAL EDM OBS	PAGE __ OF __	Bergen	27 July 84
SUBJECT		CHECKED BY	DATE
TYPICAL FIELD BOOK DATA & COMPUTATIONS		Noles	27 July 84

THE FOLLOWING DATA MUST BE FIELD RECORDED/COMPUTED FOR EACH PICES EDM OBSERVATION. NO RIGID RECORDING FORMAT IS SPECIFIED. (THE SAMPLE OBSERVATION IS TOTALLY SIMULATED.)

	$\wedge$ CFBC 992	ϕ CFBC 993	27 July 1984
			INGLIS Loch
			(Full @ 36.3)
MARK	Instrument Stand	TRIPOD	CLEAR
Instrument	AGA # XXXX	REF S/N XXX	$\wedge$ – Noles
			◊ – Bergen
Elevation	99.2198✓ m	97.6147✓ m	ϕ – Bergen
Plug Insert Office	–0.051✓ m	n/a	Check – Noles
HI (ft)/m	(0'.73) + 0.226✓ m	(5'.62) + 1.713✓ m	T – 0847 AM
Elevation	$\wedge$ 99.3948✓ m	ϕ 99.3277✓ m	

	Set 1	Set 2	Temp (°F)	Press(in Hg)
	72.1084 m	72.1086 m		
	.1087	.1087	$\wedge$ 86/87	30.12/30.15
	.1081	.1088	ϕ 85/86	
	.1083	.1085	M 86 ✓	30.1 in Hg✓
	72.1085	72.1083		
Mean (set)	72.1084✓	72.1086✓	+16✓ ppm Dialed in AGA	

Mean of Sets	72.108 m ✓	MET Corrected Slope Distance
	–0.002 m✓	System Constant (AGA # XXXX Ref/SN XXX) ~ 8 June 84 Calibration
	72.106 m✓	Corrected Slope Distance (T)

Δ elev = 0.0671✓ m

Corrected Horizontal Distance: $H = (T^2 - \Delta e^2)^{1/2}$ = 72.106✓ meters

HAD A REFLECTOR ROD BEEN USED THEN ITS SERIAL NUMBER WOULD HAVE BEEN RECORDED

Figure 12-3. Horizontal EDM observation

Appendix A
References

A-1. Required Publications

Public Law 92-367
Public Law 92-367, “The Dam Inspection Act”

Public Law 92-582
Public Law 92-582 (86 STAT. 1278), “Public Buildings--Selection of Architects and Engineers”

EFARS
Engineer Federal Acquisition Regulation Supplement

FM 5-82D/CM
Commander’s Manual: 82D
Topographic Surveyor

FM 5-232
Topographic Surveying

FM 5-233
Construction Surveying

TM 5-237
Surveying Computer’s Manual

TM 5-441
Geodetic and Topographic Surveying

TB 5-803-3-1
Guidelines for the Preparation of Automated Map Data Bases at Army Installations (Draft)

ER 405-1-12
Real Estate Handbook

ER 1110-2-100
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ER 1110-2-1150
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ER 1110-2-1200
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ER 1110-2-1806
Earthquake Design and Analysis for Corps of Engineers Projects

ER 1110-345-710
Drawings

ER 1130-2-307
Dredging Policies and Practices

ER 1130-2-315
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EP 25-1-1
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EP 310-1-6
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EM 1110-1-1000
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EM 1110-1-1002
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EM 1110-1-1005
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Appendix B
Glossary

B-1. Abbreviations

A-E	Architect-Engineer
BIH	Bureau International de l'Heure
BM	Benchmark
CCD	Charge Couple Device
CDMS	Continuous Deformation Monitoring System
CONUS	CONtinental United States
CORPSCON	CORPS CONvert
CSM	Computer Simulation Method
CTP	Conventional Terrestrial Pole
CW	Civil Works
DGPS	Differential Global Positioning System
DME	Distance Measuring Equipment
DoD	Department of Defense
EDM	Electronic Distance Measurement
EFARS	Engineer Federal Acquisition Regulation Supplement
E&D	Engineering and Design
FEM	Finite Element Method
FGCC	Federal Geodetic Control Committee
FGCS	Federal Geodetic Control Subcommittee
FGDC	Federal Geographic Data Committee
FOA	Field Operating Activity
GIS	Geographic Information System
GPS	Global Postioning System
GRS 80	Geodetic Reference System of 1980
HARN	High Accuracy Regional Networks
HI	Height of Instrument
IDT	Indefinite Delivery Type
IGLD 55	International Great Lakes Datum of 1955
IGLD 85	International Great Lakes Datum of 1985
MHW	mean high water
MLLW	mean lower low water
MLW	mean low water
MSL	mean sea level
MSL 1912	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
NATO	North Atlantic Treaty Organization
NAVD 88	North American Vertical Datum of 1988
NAVSTAR	NAVigation Satellite Timing and Ranging
NGRS	National Geodetic Reference System
NGS	National Geodetic Survey
NGVD 29	National Geodetic Vertical Datum 1929
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
OCONUS	Outside the Continental United States
OMB	Office of Management and Budget
PC	personal computer
RLR	Reference Line Ratio
RMS	root mean square
SI	International System of Units
SPCS	State Plane Coordinate System
TBM	temporary benchmark
TEC	U.S. Army Topographic Engineering Center
TM	Transverse Mercator
2DRMS	Two standard deviation root mean square
USACE	U.S. Army Corps of Engineers
USC&GS	U.S. Coast & Geodetic Survey
UTM	Universal Transverse Mercator
VERTCON	VERTical CONversion
WGS 84	World Geodetic System of 1984

B-2. Terms

Abberation of Light (Astronomic)
The apparent displacement in position of a stellar body due to the velocity of light combined with the motion of the earth itself.

Absolute Accuracy
Accuracy that is determined by a specific reference to a value.

Absolute 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.

Accidental Error
Error that is accidentally incurred in a measurement. Unlike systematic errors, accidental errors are not governed by fixed laws. The theory of probability is based

on the occurrence of these errors, which are just as likely to be positive as negative.

Accuracy
A measurement's degree of conformity or perfection. Accuracy relates to the quality of a result and is distinquished from precision which relates to the quality of the operation by which the result is obtained.

Adjustment
Adjustment is the removal of discrepancy errors. This adjustment forms a coordinated and correlated system of stations.

Agonic Line
A line, on the earth's surface, with zero magnetic declination. Equivalently, it is the locus of all points, on the earth's surface, at which magnetic north and astronomic north coincide. It is a particular case of an isogonic line.

Altimeter
An instrument that measures approximate elevations or approximate differences of elevation.

Altitude
The vertical angle between the plane of the observer's true horizon and a 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.

Apparent Time
Time that is directly determined from measurements, i.e., based on the true place of the sun. Apparent time is usually adjusted to remove effects of refraction and aberration from the measurements. Two kinds of apparent time are in common use: apparent sidereal time and apparent solar time.

Archiving
Storing of documents.

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 (0°), north and south through 90°. Astronomical latitude is the latitude which 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 which results directly from observations on celestial bodies, uncorrected for deflection of the vertical.

Astronomical Triangle
A "reference 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 light from a source outside the atmosphere. Also called astronomic refraction. Light from a star or planet passes through the entire depth of the atmosphere before reaching the surface of the earth. Refraction causes the ray to follow a curved path concave toward the surface.

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 of the two values derived by different surveys or along different routes. Usually, one value is derived by computations using the measurements made during the survey (traverse, triangulation, or trilateration); the other is an adjusted or fixed value determined by an earlier or

more precise survey or by independent, astronomical observations.

Backsight
1. (traversing) A sight on a previously established traverse or triangulation station and not the closing sight on the traverse. 2. (leveling) A reading on a rod held on a point whose elevation has been previously determined and not the closing sight of a level line.

Barometric Leveling
Determining differences of elevation from differences of atmospheric pressure observed with a barometer. By the application of certain corrections and the use of what is sometimes termed the barometric formula, a difference of atmospheric pressure at two places is transformed into a difference of elevations of those places. If the elevation of one station above a datum (as sea level) is known, the approximate elevations of other stations can be known by barometric leveling. By using barometers of special design, and including several stations of known elevation in a series of occupied stations, the accuracy of the elevations determined for the new stations is increased. Corrections are applied for temperature, latitude, index of barometer, closure of circuit, diurnal variation in atmospheric pressure, etc. Barometric leveling determined differences in height are determined by measuring the differences in atmospheric pressure at various elevations. Air pressure is measured by mercurial or aneroid barometers, or a boiling point thermometer. Although the degree of precision obtainable through this method is not as great as either of the other methods mentioned, it is a method by which to rapidly obtain relative heights at points which are at fair distances apart from each other. It is a method that has been widely used in reconnaissance and exploratory surveys where more exacting measurements will be made later or are not required.

Baseline
1. Resultant three-dimensional vector V between any two stations. Generally given in earth-centered Cartesian coordinates where $V = (\Delta x, \Delta y, \Delta z)$. 2. The primary reference line in a construction coordinate system.

Base Net
Expansion of geometric figures from a baseline to a line of the main scheme of a triangulation net.

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

Basic Control
The horizontal and vertical control used for Third-Order or higher order accuracy. It is determined in the field and permanently marked or monumented for further surveys.

Bearing
The direction of a line within a quadrant, with respect to the meridian. Bearings are measured clockwise or counterclockwise from north or south, depending on the quadrant.

Bearing Determination
The correct description of a bearing. It is not adequate to describe a line or bearing as simply northeast or southwest. All bearings meant for USACE applications need to be described as to any degrees, minutes, and seconds in the direction in which the line is progressing. The accuracy of the calculation is dependent on exact measurements. The bearing states its primary direction, north or south, first and then the angle, east or west.

Benchmark
A relatively permanent material object, natural or artificial, on a marked point of known elevation. Usually designated a BM, such a mark is sometimes qualified as a permanent benchmark or PBM to distinguish it from a temporary or supplementary benchmark designated as TBM. TBMs are marks of less permanent character and are intended to serve for only a comparatively short period of time.

Best Fit
To represent a given set of points by the corresponding points of a smooth function, curve, or surface.

Bipod
A two-legged structure used for supporting 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." This is one of many guidelines to follow for geodetic control surveys.

Blunder
A mistake or error caused by mental confusion, carelessness, stupidity, ignorance, or some other factor. Examples of blunders are: reading a horizontal circle wrong by a whole degree; neglecting to record a whole tape length in a traverse; and reversing the numerals in recording a measurement. It would also apply to the number recorded

as the result of observing on the wrong target or from the wrong control point.

Bureau International de l'Heure (BIH)
Coordinates the measurements of time by national observatories and provides an internationally acceptable, common time. It is also responsible for maintaining the international atomic second, i.e., providing to users a unit of time, the second, against which other standards can be calibrated. As part of its function, it calculates the position of the earth's axis of rotation with respect to points on the earth and changes in the earth's rate of rotation. 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. The timekeeping portion of the BIH was transferred to the Bureau International des Poids et Mesures (BIPM).

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 U.S., 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 an instrument of nearly correct measurements. The correct value is established either by definition or by measurement with a device which has itself been calibrated against a device considered correct. Frequently used instruments are usually calibrated using a measuring device (a working standard) which has itself been calibrated. Working standards are calibrated by comparing them with another calibrated set called laboratory standards, and these in turn are calibrated by comparison with the primary standard. Also called standardization. However, that term is now used to mean the imposition of a standard on otherwise diverse processes and products.

Cartesian Coordinates
A system with its origin at the center of the earth and the x- and y-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 N straight lines (called the axes) intersecting at a common point (called the origin). The nth coordinate ($1 \leq n \leq N$) of a point is the distance between that point and the hyperplane determined by all axes but the nth, and measured parallel to the nth axis. Alternatively, a set of N families of N-1 dimensional hyperplanes such that members of the same family have no line in common, while members of different families intersect in one and only one line. The coordinates of a point are then the set of values of the parameters determining the N hyperplanes passing through that point. The units in which distances are measured need not be the same along all the axes, and the axes need not intersect at right angles. A Cartesian coordinate system (CCS) for which the units of distance are different in different directions is sometimes erroneously called an affine CCS. If all the axes intersect at right angles, the system is called a rectangular CCS or simply a CCS. Otherwise, it is called an oblique CCS. Distances are usually measured from the hyperplane to the point and are assigned a positive or a negative value according to some specified convention.

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
1. The 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. 2. The meridian used as y-axis in computing tables for a State Plane Coordinate system. The central meridian of the coordinate system usually passes close to the geometric center of the region or zone for which the tables are computed but, to avoid using negative values, is given a large positive value which must be added to all x-coordinates. 3. That

line, on a graticule, which represents a meridian and which is an axis of symmetry for the geometric properties of the graticule.

Chain
Equal to 66 feet or 100 links. The unit of length prescribed by law for the survey of the U.S. public lands. One acre equals 10 square chains. The chain derives its name from the Gunter's chain, which was widely used in early surveys and had the form of a series of links connected together by rings.

Chained Traverse
Observations and measurements performed with tape.

Chaining
Measuring distances on the ground with a graduated tape or with a chain. The term taping is now preferred if a tape is used. Although the chain has been superseded by the graduated tape for making land and other surveys, the term "chaining" has continued in use in some surveying organizations and in places where reference is to surveys of the public lands of the U.S. For the corresponding operation in other surveys, the term taping is preferred. In chaining, the persons who mark the tape's ends are called chainmen.

Chart Datum
For referring soundings of a datum to a 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 U.S., its territories, Commonwealth of Puerto Rico, and Trust Territory of the Pacific Islands.

Chi-square Testing
Test whether the classification of data can be ascribed to chance or to some underlying law.

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 in some measured function of location from the measurements or calculations carried out for points along a line that begins and ends at the same point.

Circumpolar Star
A star in any given latitude which never goes below the horizon; hence its polar distance must be less than the given latitude. In astronomy only those stars with a polar distance of less than 10° are considered in practical problems.

Clarke 1866 Ellipsoid
A rotational ellipsoid with: semi-major axis = 20,926,062 feet (6,378,206.4 m), semi-minor axis = 20,855,121 feet (6,356,583.8 m), flattening (derived) = 1/294.978. This ellipsoid has been in use since 1880, in the U.S., for calculating triangulation. The metric values were calculated using Clarke's legal meter of 1866, and were used in the past in tables based on this ellipsoid. Values based on the value 0.304 800 47 for the ratio of the foot to the meter:

semi-major axis	6,378,274	m
semi-minor axis	6,356,650	m.

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

Coefficient of Refraction
The ratio at the point of observation to the angle, at the center of the earth between the point of observation and the point observed.

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

Collimation Error
The angle between the actual line of sight through an optical instrument and the position the line would have in a perfect instrument.

Compass Rule
The correction to be 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. Also called

Bowditch's rule. The compass rule is used when it is assumed that the misclosure results as much from errors in measured angles as from errors in measured distances. Care must be used in applying the rule because there is no general agreement on the sign to be used for the misclosure; the corrections must be applied with signs determined by the particular conventions in use.

Compensator
Keeps the line of sight horizontal regardless of the tilt (within limits) of the rest of the instrument. It consists of at least one optical element (prism, mirror, or train of prisms or mirrors) suspended in such a way as to keep its orientation with respect to the direction of gravity fixed. It may be suspended by fine wires of invar or by flexible tapes, or it may be attached as the bob of a pendulum. Then gravity keeps the component from rotating with the rest of the instrument when the instrument is tilted slightly. The arrangement of rotating and nonrotating components is such that a horizontal ray entering the objective lens of a tilted telescope is deflected by the compensator just enough that it passes through the center of the reticle and thence through the ocular to the observer. The compensator of most self-leveling instruments can compensate for about 10' of tilt in the instrument. It replaces the sensitive spirit level formerly attached to the telescope and thereby does away with the need to compensate manually for tilt. A leveling instrument equipped with a compensator is frequently referred to as an automatic level. However, the terms compensator leveling instrument or self-leveling instrument are preferred.

Confidence Level
Accuracy based on a known distribution function; e.g., the normal distribution function or bivariate normal distribution function. Errors are stated as some percentage of the total probability of 100 percent; e.g., a 90 percent confidence level.

Conformal
Map projection that holds true shapes at any points but exaggerates the area.

Contour
An imaginary line on the ground with all points thereof at the same elevation above or below a specified surface of reference. The definition is illustrated by the shore line of an imaginary body of water whose surface is at the elevation represented by the contour. A contour forming a closed loop round lower ground is called a depression contour. Contour should not be confused with contour line; the latter is the term for a line drawn on a map. However, the distinction is ignored by some writers.

Control
1. The coordinated and correlated dimensional 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. 2. A collective term for a system of marks or objects on the earth or on a map or a photograph whose positions or elevation, or both, have been or will be determined.

Control Densification
The addition of control throughout an established control region.

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 used in other dependent surveys. The term 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 or graphic process of obtaining a modified set of coordinates. Transformation is completed

through some combination of rotation of coordinate axes at their point of origin, modification of scale along coordinate axes, or change of the size of geometry of the reference space.

CORPSCON
(Corps Convert) Software package (based on NADCON) capable of performing transformations between NAD 83 and NAD 27 geographical coordinates, CORPSCON also converts between State Plane Coordinates System (SPCS), geographical coordinates, and Universal Transverse Mercator (UTM) coordinates; thus eliminating several steps in the total process of converting between grid coordinates on NAD 27 and NAD 83. Inputs can be either in geographic, UTM, or SPCS coordinates. This program can also be used to convert between grid and geographic coordinates on the same datum.

Crandall Method
The 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 corrections distributed among the measurements of distance through the method of weighted least-squares.

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

Culmination
The instant when any point on the celestial sphere is on the meridian of an observer. When it is on that half of the meridian containing the zenith, it is called the upper transit; when it is on the other half, it is called the lower transit.

Curvature
1. The rate at which a curve deviates from a straight line. 2. The vector dt/ds, where t is the vector tangent to a curve and s is the distance along that curve. 3. The rate at which a curved surface deviates from a flat surface in a particular direction.

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). Declination, conventionally denoted by δ, is measured by the arc of hour circle between the celestial body and the equator; it is positive when the body is north of the equator and negative when south of it. It corresponds to latitude on the earth and with right ascension forms a pair of coordinates which defines the position of a body on the celestial sphere.

Deflection of the Vertical
The angular difference, at any place, between the upward direction of a plumb line (the vertical) and the perpendicular (the normal) to the reference spheroid. This difference seldom exceeds 30 seconds. Often expressed in two components, meridian and the prime vertical.

Deflection Traverse
Direction of each course measured as an angle from the direction of the preceding course. A deflection traverse is usually an open traverse.

Deformation Monitoring
Observing the conditions of dams. It is necessary to keep accurate records of normal occurrences to differentiate between possible hazards and normal changes.

Departure
The orthogonal projection of a line, on the ground, 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. It is east, or positive, and is sometimes called the easting, for a line whose azimuth or bearing is in the northeast or southeast quadrant. It is west, or negative, and is sometimes called the westing, for a line whose azimuth or bearing is in the southwest or northwest quadrant. It is abbreviated as dep. in notes.

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. The process actually involves the measurement of the difference in ranges between the satellites and two or more ground observing points. The basic principle is that the absolute positioning errors at the two receiver points will be approximately the same for a given instant in time. The resultant accuracy is extremely high. This type of positioning can be performed in either a static or kinematic mode.

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

Direction
1. 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. They are usually collected into a single list of directions, with the direction of 0° placed first and the other directions arranged in order of increase clockwise. 2. 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. Two lines must be specified for the first definition to be valid; only one line need be specified for the second meaning. 3. An indication of the location of one point with respect to another without involving the distance between the two points. It is usually thought of as a short segment (of a straight line between the two points) having one end at one of the points and having an arrow-like symbol at the other end. Note that a direction is not an angle.

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 or by a hinge that can be manipulated by a micrometer screw.

Dynamic Height
That vertical distance between the point P_n on the earth's surface and the point P_o on the geoid which is given by the function $\int (g/\tau_{45})\, dH$ (the integral being taken from P_o to P_n), in which g is the value of gravity acceleration along the path of integration, τ_{45} is the value of gravity at 45° latitude, as calculated from a standard gravity formula, and dH is the increment of vertical distance along the path of integration. Also called dynamic number.

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

Earth Curvature
The curvature of an ellipsoid representing the earth. In surveying over short distances, if the earth's curvature needs to be taken into account at all, representing the earth by a sphere is adequate.

Easting
The distance eastward (positive) or westward (negative) of a point from a particular meridian taken as reference. (It is common practice to use positive westings instead of negative eastings.)

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

Ecliptic
The intersection of the plane of the earth's orbit with the celestial sphere.

Electronic Distance Measurement (EDM)
Pulsing of phase comparison to determine a distance.

Elevation
The height of an object above some reference datum.

Ellipsoid
Formed by revolving an ellipse about its minor axis. An ellipsoid is defined by the length of its semimajor axis a and its flattening f, where: $f = (a\text{-}b)/a$ and b = length of the semiminor axis. The most commonly used 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). Prior to January 1987, GPS operated with reference to WGS 72. Since January 1987, it has been referenced to WGS 84. For most purposes, GRS 80 and WGS 84 can be considered identical.

Ellipsoid Height
The elevation h of a point above or below the ellipsoid.

Elongation
That point in the apparent movement of a circumpolar star when it has reached the extreme position east or west of the meridian.

Emulsion
A suspension of either light-sensitive silver salts, Diazos, or photopolymers, in a colloidal medium which is used for coating films, plates, and papers.

Ephemeris Time
The uniform measure of time defined by the laws of dynamics and determined in principal from the oribital motions of the planets, specifically in the orbital motion of the earth as represented by Newcomb's "Tables of the Sun."

Equation of Time
The algebraic difference in hour angle between apparent solar time and mean solar time, usually labeled + or - as it is to be applied to mean solar time to obtain apparent solar time.

Equinox
One of the two points of intersection of the ecliptic and the celestial equator, occupied by the sun when its declination is 0°.

Error
1. The difference between the measured value of a quantity and the theoretical or defined value of that quantity: ε(error) ≡ y (measured) - y (theoretical). 2. The difference between an observed or calculated value of a quantity and the ideal or true value of that quantity. Logically, definitions (1) and (2) are distinctly different. In practice, they are equivalent except for the second definition's allowing calculated values to be used instead of measured values.

Error Ellipse
Containing a specified percentage of randomly distributed points or having a specified value for the quadratic form $[(\xi / \sigma_x)^2 - 2\rho (\xi / \sigma_x) (\eta / \sigma_y) + (\eta / \sigma_y)^2] / (1 - \rho^2)$, in which σ_x and σ_y are the standard deviations of x and y, respectively, and ρ is the correlation coefficient. Also called an ellipse of error and contour ellipse. Unless specifically stated otherwise, the distribution is assumed to be bivariate Gaussian.

Error of Closure
Difference in related measurements of the true or fixed value of the same quantity.

Exterior Angle
One of the angles lying outside a pair of parallel lines intersected by a third line.

External Accuracy
A measure of the closeness of a set of values, measured or calculated, to the true value.

Finite Element Method
Obtaining an approximate solution to a problem for which the governing equations and boundary conditions are known. The method divides the region of interest into numerous, interconnected subregions (finite elements) over which simple, approximating functions are used to represent the unknown quantities. The basic idea originated in the early 1900's as a method of solving for strains in bridges and buildings. It was further developed for analyzing the structure of aircraft in the early 1950's. The method, supported by rigorous mathematical theory, has now been applied to problems in heat flow, hydraulics, geodynamics, and other disciplines.

Fixed Elevation
Adopted, either 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
1. An observation of the distance and direction to the next instrument station. 2. (transit traverse) A point set ahead to be used for reference when resetting the transit or line or when verifying the alinement. 3. (leveling) The reading on a rod that is held at a point whose elevation is to be determined.

Free Adjustment
1. An adjustment in which the number of unknowns is greater than the number of independent equations relating the unknowns to observations. Such an adjustment has, in general, no unique solution. More or less arbitrary conditions must therefore be imposed on the unknowns to obtain a unique solution. 2. An adjustment in which the number of unknowns is greater than the number of independent equations relating the unknowns to observations, but there are just enough conditions imposed on the unknowns to produce a unique solution. 3. The adjustment of a network containing no fixed quantities. 4. An adjustment in which the rank of the normal matrix, exclusive of that part associated with Lagrangian multipliers, is equal to the order of the normal matrix. 5. An adjustment of geodetic coordinates in which, if the number of points with unknown coordinates is N, there are 3N unknowns in 3-space (or 2N unknowns in 2-space) and there are 6 (or 3) condition equations. Also called adjustment using minimal constraints or inner adjustment.

Frequency
The number of complete cycles per second existing in any form of wave motion.

Geocentric Coordinates
1. One of a set of three coordinates, of a point, in a geocentric coordinate system. The coordinates are commonly represented by (λ, ϕ', r) where λ, the longitude, is the angle from a reference plane through the polar axis to the plane containing the polar axis and the point; ϕ', the geocentric latitude, is the angle from the equatorial plane to the radius vector to the point; and r, the geocentric distance or geocentric radius, is the distance of the point from the center. The letter ψ is frequently used instead of ϕ'. Note that the origin (intersection of polar axis and equatorial plane, or center of reference ellipsoid if one is used) need not be at the earth's center of mass or geometric center. The complete set of three geocentric coordinates is also called a geocentric position. 2. One of a set of coordinates designating the location of a point by means of (a) the angle from the plane of the celestial equator to a line from the center of the earth to the point and (b) the angle from the plane of a selected, initial geodetic meridian to that line. The first angle is called the geocentric latitude. The term geocentric longitude is not used for the second angle because that angle is the same as the geodetic longitude. This definition properly applies only to the exceptional case where the plane of the equator passes through the center of the earth and the polar axis of the reference ellipsoid coincides with the earth's polar axis. 3. The longitude and latitude of a point (on the earth's surface) relative to the center of the earth. The earth's center of mass is usually meant.

Geodesic Line
Shortest distance between any two points on any mathematically defined surface. A geodesic line is a line of double curvature, and usually lies between the two normal section lines which the two points determine. If the two terminal points are in nearly the same latitude, the geodesic line may cross one of the normal section lines. It should be noted that, except along the equator and along the meridians, the geodesic line is not a plane curve and cannot be sighted over directly. However, for conventional triangulation the lengths and directions of geodesic lines differ inappreciably from corresponding pairs of normal section lines.

Geodesy
Determination of the size and figure of the earth (geoid) 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 (geoid) have been considered in position computations.

Geodetic Coordinates (ϕ, λ, H)
Angular latitudinal and longitudal coordinates, usually referenced to some defined ellipsoid of revolution.

Geodetic Height
The perpendicular distance from a specified ellipsoid (reference ellipsoid) to the point of interest. Note that geodesists distinguish sharply between geodetic height, which is measured along the perpendicular to the reference ellipsoid, and elevation, which is measured along the vertical to the reference surface. Conventionally, h is used for geodetic heights, H for elevations.

Geodetic Latitude
The angle which the normal at a point on the reference spheroid makes with the plane of the geodetic equator. Geodetic latitudes are reckoned from the equator, but in the horizontal-control survey of the U.S. they are computed from the latitude of station Meades Ranch as prescribed in the North American Datum of 1927.

Geodetic Leveling
The observation of the differences in elevation by means of a continuous series of short horizontal lines of sight. 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 or pendulum apparatus (compensator) that senses the vertical/ horizontal directions. This is the most accurate method of determining elevations.

Geodetic Longitude
The arbitrarily chosen angle between the plane of the geodetic meridian and the plane of an initial meridian. A geodetic longitude can be measured by the angle at the pole of rotation of the reference spheroid between the local and initial meridians, or by the arc of the geodetic equator intercepted by those meridians. In the U.S., geodetic longitudes are numbered from the meridian of Greenwich, but are computed from the meridian of station Meades Ranch as prescribed in the North American Datum of 1927. A geodetic longitude differs from the corresponding astronomical longitude by the amount of the prime vertical component of the local deflection of the vertical divided by the cosine of the latitude.

Geodetic North

1. That direction, along a tangent to a meridian on an oblate, rotational ellipsoid representing the earth, which is approximately that of astronomic north. Also called true north, although that term is usually applied only to astronomic north. 2. The direction of a vector parallel to the minor axis of an oblate, rotational ellipsoid and directed approximately toward astronomic north.

Geodetic Reference System of 1980 (GRS 80)

The following set of numbers adopted in 1979 by the International Union of Geodesy and Geophysics:

a	6 378 137 meters
GM	98 600.6 x 10^9 m^3/s^2
J_2 (- C_{20})	-0.001 082 63
ω	729 211.5 x 10^{-11} radians/s.

a is the length of the semimajor axis of an equipotential, rotational ellipsoid, GM is the product of the gravitational constant G by the mass M of the earth, and C_{20} is the coefficient of the second-degree Legendre function in the representation V of the earth's gravitational potential $V = (GM/a)\ \Sigma_n\ [(a/r)\ n+1\ \Sigma_n\ \{(C_{nm} \cos m\lambda + S_{nm} \sin m\lambda)\ P_{nm} (\sin \phi)\}]$, in which r is geocentric distance, ϕ is geocentric latitude, and λ is geodetic longitude. ω is the angular rate of rotation of the ellipsoidal body.

Geographic Coordinates

1. A general term for either a geodetic or an astronomic coordinate. Also called a terrestrial coordinate. The pair of geographic coordinates is also called a geographic location or a geographic position. 2. One of a pair of coordinates which specify the angle between a specified line and a meridional plane and the angle between that line and an equatorial plane.

Geoid

An equipotential surface approximating the earth's surface and corresponding with mean sea level in the oceans and its extension through the continents. In other words, the geoid would coincide with the surface to which the oceans would conform over the entire earth if the oceans were set free to adjust to the combined effect of the earth's mass attraction and the centrifugal force of the earth's rotation.

GPS (Global Positioning System)

The NAVSTAR satellites in six different orbits, five monitor stations and the user community.

Gravimeter

Registers variations in the weight of a constant mass when the mass is moved from place to place on the earth. It is subjected to the influence of gravity at those places.

Gravity

Viewed from a frame of reference fixed in the earth, acceleration imparted by the earth to a mass which is rotating the earth. Since the earth is rotating, the acceleration observed as gravity is the resultant of the acceleration of gravitation and the centrifugal acceleration arising from this rotation and the use of an earthbound rotating frame of reference. It is directed normal to sea level and to its geopotential surfaces.

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. Observations to maintain the Greenwich meridian as reference are no longer made at the Airy transit or, for that matter, at the Greenwich Observatory. Instead, they are made (1986) at Herstmanceaux Observatory, England, and the observations made there are reduced, by calculation, to the equivalent values at the Airy transit. A meridian close to that of Greenwich but used as a reference for determining time internationally is maintained by the Bureau International de l'Heure. Although the definitions of the Greenwich meridian and of the meridian used as reference for times are valid, neither provides a fixed reference for either longitude or time. The meridian of Greenwich changes its position with respect to meridians at other observatories because of geological changes in the region surrounding the Greenwich Observatory. The meridian provided by the Bureau International de l'Heure is affected by the errors in time signals from the stations monitored by the Bureau.

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. In the State Plane Coordinate System established by the U.S. Coast and Geodetic Survey, grid azimuths were reckoned from south (0°) clockwise through 360°. To reduce geodetic azimuth to grid azimuth, a quantity $[y_2 - y_1)\ (2x' + x_2')]/(6\ r_o \sin 1'')$ should be subtracted from the geodetic azimuth if the line is more than 7 km long and if full accuracy is wanted. (x_i', y_i are the coordinates of the points whose azimuth is wanted, and the denominator is a constant for the coordinate system of a particular State and is given in the tables of plane rectangular coordinates for that State). 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
1. That line, in a grid on a map, parallel to the line representing the central meridian or y-axis. 2. That line, in a rectangular, Cartesian coordinate system applied to a map, parallel to the line representing the y-axis or central meridian.

Gulf Coast Low Water Datum (GCLWD)
Used as chart datum for the coastal waters of the Gulf Coast of the U.S. It is defined as mean lower low water when the tide is of the mixed type and as mean low water when the tides are diurnal. The datum was abandoned by the National Ocean Survey in 1980 and replaced by mean lower low water for the region involved.

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. Invented by the English astronomer Edmund Gunter about 1620, it is the basis for the chain and link, units of length used in surveying the public lands of the USA. The original form of the chain has been replaced by metallic tapes or ribbons graduated in links and chains, but the new forms are still called chains.

Gyrotheodolite
A built-in gyroscopic compass to measure angles with respect to that North indicated by the compass. Also called a gyro-azimuth theodolite, gyroscopic meridian-indicating instrument, gyro-meridian indicating instrument and gyroscopic theodolite. The instrument is particularly useful in mines and other places (such as the far north) where direction is difficult to establish by other means.

Hachures
Portraying relief by short, wedge-shaped marks radiating from high elevations and following the direction of slope to the lowland.

Heliotrope
One or more plane mirrors so mounted at the station being sighted upon, that the sun's rays can be reflected to any one observing station.

Histogram
A graphical representation of a frequency function. The frequency of occurrence is indicated by the height of a rectangle whose base is proportional to the interval within which the events occur with the indicated frequency.

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

Horizontal Refraction
A natural error in surveying. The result of the horizontal bending of light rays between a target and an observing instrument causes this refraction. Usually caused by the differences in density of the air along the path of the light rays, resulting from temperature variations.

Hour Angle
Angular distance west of a celestial meridian or hour circle; the arc of the celestial equator, or the angle at the celestial pole, between the upper branch of a celestial meridian or hour circle and the hour circle of a celestial body or the vernal equinox, measured westward through 360°.

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

Index Error
1. A systematic error caused by misplacement of an index mark or zero mark on an instrument having a scale or vernier, so that the instrument gives a nonzero reading when it should give a reading of zero. This is a constant error. 2. The distance upward (or downward) from the foot of a leveling rod (the lowest horizontal surface) to the nominal origin (theoretical zero) of the scale.

Indirect Leveling
The determination of differences of elevation from vertical angles and horizontal distances, as in *trigonometric leveling*; comparative elevations derived from values of atmospheric pressure determined with a barometer, as in *barometric leveling*; and elevations derived from values of the boiling point of water determined with a hypsometer, as in *thermometric leveling.*

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 m.

International Great Lakes Datum of 1955 (IGLD 55)
Vertical reference datum used in the Great Lakes and their connecting waterways.

International Great Lakes Datum of 1985 (IGLD 85)
The adjustment of IGLD 55 used data collected from 1982-1988. The most significant change is in elevations assigned to water levels. The water levels established for Canadian zoning restrictions and U.S. flood insurance purposes will not change. The implementation and publication of IGLD 85 occurred in January 1992, and should be used for at least 25 years.

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, changed in 1983.

Interpolation Method
1. Determination of an intermediate value between given values, from some known or assumed rate or system of change of values between the given ones. Equivalently, determination of the value of a function $y = f(x)$ for some arbitrarily specified value of x, given values (y_1, x_1) and (y_2, x_2) such that the arbitrary value of x lies between the two given values. 2. The process or technique of obtaining an estimate of the value of a quantity at a specified time t, given that data useful for obtaining the estimate are available for times both before and after t. This definition seems to be part of the jargon of control engineering, in which it is called smoothing.

Intersection
Determining the horizontal position of a point by observations from two or more points of known position. Thus measuring directions 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 can see each other in a survey net.

Invar
An alloy of iron containing about 64% iron, 36% 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). It was invented by C. Guillaume of the International Bureau of Weights and Measures (Paris). It is used wherever a metal does not change its dimensions appreciably with temperature as desired. For this reason, it or a similar alloy has replaced steel in surveyor's tapes and wires, and particularly in those tapes and wires used for measuring geodetic baselines. It is also used for the scale of some leveling rods, in first-order leveling instruments, and in pendulums.

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. Also called an isogonal.

Kinematic Survey Methods
Requires two receivers recording observations simultaneously. It is often referred to as dynamic surveying. As in stop-and-go surveying, the reference receiver remains fixed on a known control point while the rover receiver collects data on a constantly moving platform. Unlike stop-and-go surveying, kinematic surveying techniques do not require the rover receiver to remain motionless over the unknown point. The observation data are later postprocessed by computer to calculate relative vector/coordinate differences to the roving receiver.

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. Thus Laplace correction = $(\lambda_A - \lambda_G) \sin \phi_G$.

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 by determining the corrections so that the sum of the squares of the corrections is a minimum.

Length of Closure
The distance defined by the equation:
$[(\text{closure of latitude})^2 + (\text{closure of departure})^2]^{0.5}$

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 U.S. 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 (benchmark).

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

Line of Sight
1. The straight line between two points. This line is in the direction of a great circle, but does not follow the curvature of the earth. 2. The line extending from an instrument along which distant objects are seen, when viewed with a telescope or other sighting device.

Local Coordinate System
The origin is within the region being studied and used principally for points within that region.

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

Local Hour Angle
Angular distance measured on the celestial equator between the celestial meridian and the hour circle that passes through the object. The local hour angle represents physically the amount of rotation of the celestial sphere, since the object was last on the observer's celestial meridian, and is always measured westward 0° to 360° from the celestial meridian.

Loop
A pattern of measurements in the field, so that the final measurement is made at the same place as the first measurement.

Loop Traverse
A closed traverse that starts and ends at the same station.

Lovar
An alloy having a coefficient of expansion between that of steel and that of invar but costing considerably less than invar.

Magnetic Bearing
The angle with respect to magnetic north or magnetic south and stated as east or west of the magnetic meridian, e.g., N 15°E.

Magnetic Meridian
1. The vertical plane in which a freely suspended, symmetrically magnetized needle, influenced by no transient, artificial, magnetic disturbance, will come to rest. 2. The curve, on the earth's surface at all points of which the vertical plane described in the preceding definition is tangent to the curve. 3. The direction, at any point, of the horizontal component of the earth's magnetic field.

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 nonexistent 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, or the average ratio of specified distances, on a map to the corresponding distance or distances on the ellipsoid used in making the map (i.e., used in the map projection).

Mean Angle
Average of the angles.

Mean Lower Low Water (MLLW)
The average height of all lower low waters recorded over a 19-year period. It is usually associated with a tide exhibiting mixed characteristics.

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 U.S., is based on tidal observations over a number of years at various tide stations along the coasts.

Mean Sea Level Datum of 1912 (MSL 1912)
MSL 1912 was the last adjustment before the Sea Level Datum of 1929 was established and only included level lines of the U.S.

Meridian Angle
Angular distance east or west of the local celestial meridian, the arc of the celestial equator, or the angle at the celestial pole, between the upper branch of the local celestial meridian and the hour circle of a celestial body, measured eastward or westward from the local celestial meridian through 180°, and labeled E or W to indicate the direction of measurement.

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

Micrometer
1. In general, any instrument for measuring small distances or angles very accurately. 2. 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 hairlike filament, the micrometer is called a filar micrometer.

Misclosure
1. In general, the amount by which a value (of a quantity) obtained by surveying fails to agree with a value (of the same quantity) determined, e.g., by an earlier survey, an arbitrarily assigned starting value, or from theory. Also called closure, closing error, and error of closure. 2. In leveling, the amount by which the two values for the elevation of the same benchmark, derived by different surveys, by the same survey made along two different routes, or by independent measurements, fail to exactly equal each other. The misclosure may occur in a line of leveling beginning and ending on different benchmarks whose elevations are held fixed, or beginning and ending on the same benchmark. 3. (traversing) The amount by which a value for a component of the location (of a traverse station) obtained by computation fails to agree with another value for the component as determined by a set of measurements over the same or a different route. Also called closure, error in closure, traverse error in closure, closure in position, total closure, and misclosure in position. The traverse may run between two stations whose locations are held fixed or it may begin and end at the same station. In either case, there are two values for the location of the final station: one known before the traverse was computed and the other obtained by computation using the measurements made on the traverse. The difference between these is the misclosure. It may be resolved into misclosure in latitude, misclosure in longitude (departure), or both. Although the definition calls for the difference (computed value minus measured value), in practice the difference may be taken either way and corrections applied in an ad hoc manner.

Monument
Indication of the position on the ground of a survey station. In military surveys, the term monument usually refers to a stone or concrete station marker containing a special bronze plate on which the exact station point is marked.

NADCON
The National Geodetic Survey developed the conversion program NADCON (North American Datum Conversion) to provide consistent results when converting 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 on the terrestrial sphere directly beneath the observer and directly opposite to the zenith; the lowest point.

National Geodetic Vertical Datum 1929 (NGVD 29)
Adopted as the standard geodetic datum for heights, based on an adjustment holding 26 primary tide stations in North America fixed. The latest general adjustment is the NGVD 29. Portions of the upper Mississippi River are referenced to the previous (1912) general adjustment. A new readjustment is currently in progress, and will be termed the North American Vertical Datum of 1988 (NAVD 88) when completed. NGVD is not the same as mean sea level.

National Map Accuracy Standards
Set by the U.S., specifying the accuracy required of topographic maps published by the U.S. at various scales. The standards for horizontal accuracy specify that for maps at scales larger than 1:20,000, 90% of all well-defined features (with the exception of those unavoidably displaced by exaggerated symbolization) shall be located within 1/30 inch (0.85 mm) of their geographic locations as referred to the graticule, while for maps published at scales of 1:20,000 or smaller, 1/50 inch (0.50 mm) is the criterion. The standards for vertical accuracy specify that 90% of all contours and elevations determined from contour lines shall be accurate to within one-half of the basic contour interval. Errors (of contours and elevations) greater than this may be decreased by assuming a horizontal displacement within 1/50 inch (0.50 mm).

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. The epoch is designated by giving the year the period began and the year it ended, e.g., National Tidal Datum Epoch of 1941 through 1959.

Network
Interconnected system of points. Although there are different networks of horizontal and vertical control, they are all based on data observed with instruments oriented to the earth's gravity field.

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

North American Datum of 1927 (NAD 27)
The initial point of this datum is located at Meades Ranch, Kansas. Based on the Clarke spheroid of 1866, the geodetic positions of this system are derived from a readjustment of the triangulation of the entire country, in which Laplace azimuths were introduced.

North American Datum of 1983 (NAD 83)
Covers all of North America and extends into Greenland. NAD 83 was developed by coordinated efforts between the countries covered by the system, and essentially completed by 1988. The datum of the North American Datum 1983 geodetic system is, for practical purposes, equivalent to that of the World Geodetic System 1984. Coordinates of points in the system are not, however, the same because they are determined by including in the adjustment besides data from the TRANSIT (Navy Navigation Satellite System), data from traverses, triangulation, and the GPS.

North American Vertical Datum of 1988 (NAVD 88)
Defined by only one tidal station in Quebec, Canada. NAVD 88 is the new datum and before its advent, all leveling was referred to NGVD 1929.

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

Observed Angle
May or may not have been corrected for local conditions only, at the point of observations.

Observer's Meridian
A celestial meridian passing through the zenith at the point of observation and the celestial poles.

Occultation
Applied to a geodetic survey technique which employs the principle of occultation where repeated observations are made on an unknown position, accurately timed with similar observations at another unknown station, and mathematically reducing these data to determine the exact geodetic position of the unknown stations.

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 downward. 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. An optical plummet is not affected by wind and is therefore superior to the plumb line in this respect. Most modern theodolites have an optical plummet built into the base of the instrument, so that the upright section of the optical axis of the plummet coincides with the vertical axis of the theodolite. The eyepiece is usually located near the base.

Order of Accuracy
Defines the general accuracy of the measurements made in a survey. The order of accuracy of surveys is divided into four classes labeled: first order, second order, third order, and fourth or lower order.

Origin
1. That point, in a coordinate system, which has defined 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. In surveying, when a plane rectangular coordinate system is used, the coordinates of the origin are often given large, positive values. These values are called false easting (or false westing) and false northing (or false southing). 2. The point to which the coordinates (0,0,0,....0) are assigned, regardless of the location of that point with respect to the axes of the coordinate system. 3. The point from which coordinates of other points in the coordinate system are reckoned. 4. (of a datum) A point whose coordinates are defined and are part of a datum (usually a geodetic datum). The origin is usually a survey station.

Orthometric Height
The elevation H of a point above or below the geoid. A relationship between ellipsoid heights and orthometric heights is obtained from the following equation: $h = H + N$ where h = ellipsoidal height, H = orthometric height, and N = geoidal height.

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.

Personal Equation
The time interval between the sensory perception of a phenomenon and the motor reaction thereto. A personal equation may be either positive or negative, as an observer may anticipate the occurrence of an event, or wait until he actually sees it occur before making a record. This is a systematic error, treated as the constant type.

Personal Error
Caused by an individual's personal habits, his inability to perceive or measure dimensional values exactly, or by his tendency to react mentally and physically in a uniform manner under similar conditions.

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. Also called rarely and erroneously, metrical photography. A modifier is customarily used with the term to indicate the type of radiation causing the photographic image when this is not light, e.g., X-ray photogrammetry, infrared photogrammetry, and acoustic photogrammetry. The principal application of photogrammetry is to the mapping of the earth's surface, but it is also used for mapping the surface of other bodies in the solar system, for recording the geometry of architectural and archaeological objects, for making anthropometric measurements, and in many other sciences and technologies.

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, i.e. fire hydrant.

Plumb Line
The line of force in the geopotential field. The continuous curve to which the direction 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.

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
Used for a specific project.

Project 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 allows every point on the first surface to correspond exactly to one point on the second surface. A projection differs from a map projection in that the latter deals only with situations in which one surface is an ellipsoid and the other a developable surface.

Quadrangle
Consisting of four specified points and the lines or line segments on which they lie. The quadrangle and the quadrilateral may define the same shape. They differ in that the quadrangle is defined by four specified points, the quadrilateral by four specified lines or line-segments.

Quad Sheet
Slang for a USGS 1:24,000-scale map.

Random Error
Produced by irregular causes whose effects upon individual measurements are governed by no known law connecting them with circumstances and which therefore can never be subjected to computation a priori. Sometimes called an irregular error or, more commonly, an accidental error. The theory of elemental errors assumes that a random error is assumed to be composed of an infinite number of independent, infinitesimal errors, all of equal magnitude, each as likely to be positive as negative. In practice, a random error is composed of an indefinitely large number of finitely small errors, each as apt to be positive as negative. With these assumptions, probabilities can be associated with the results of the method of least squares, and it is to the elimination of random errors only that the method of least squares properly may be applied.

Range Pole
A simple rod, round or octagonal in section, 6 to 8 feet long, 1 inch or less in diameter, fitted with a sharp-pointed shoe of steel and usually painted alternately in red and white bands at 1-foot intervals. It may be made of wood or metal. It is used to line up a point of a survey or to show the observer at the theodolite the location of a point on the ground.

Readings
The number obtained by noting and/or recording that number which an instrument indicates is a result of the measurement.

Real-time
1. Represented as a real quantity rather than as an imaginary quantity. This is usually the case in relativistic mechanics; the other three, space-like coordinates are then imaginary quantities. However, many scientists treat the space-like coordinates as real and the time as imaginary. 2. Very close to the instant of observation or other activity. 3. Reported or recorded at the same time as they are

happening. 4. Absence of delay in getting, sending, and receiving data.

Reciprocal Leveling
Measuring vertical angles or making rod readings from two instrument positions for the purpose of compensating for earth curvature, the effects of atmospheric refraction, and some instrument misadjustments.

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

Rectification (of datums)
Producing, from a tilted or oblique photograph, a photograph from which displacement due to tilt has been removed. Also called transformation. A photograph may be rectified optically, graphically, or mathematically. The optical method is the most common. The photograph is given the same orientation with respect to an unexposed photographic film or plate as the ground plane had to the photograph when the picture was taken. The image on the photograph is then projected onto the emulsion to reverse, in effect, the original process by which the photograph was taken. Graphical rectification involves plotting selected points from the photograph onto another sheet, using a graphical method of correcting for displacement because of tilt. Neither method compensates for radial displacement caused by relief. Mathematical rectification uses a computer as intermediary between the original photograph and the rectified image, correcting not only for nonverticality of the photograph but also, in suitable cases, for relief-caused displacement.

Redundant Measurements
Overabundant or excess measurements.

Reference Meridian, Magnetic
Based on the magnetic pole.

Reference Meridian, True
Based on the astronomical meridian.

Reference Point
Used as an origin from which measurements are taken or to which measurements are referred. Also called a datum point.

Refraction
The bending of sonic or electromagnetic rays by the substance through which the rays pass. The amount and direction of bending are determined by a characteristic of the substance called the refractive index. (Relativistic bending by a gravitational field is not refraction.)

Rejection Criterion
Set of rules and guidelines established for rejecting information, basically used in adjustments.

Relative Accuracy
1. The square root of the average of the sum of the squares of the differences between (a) a set of measured or calculated values and (b) a set of corresponding correct values, divided by the average value of the set; i.e., relative accuracy is equal to the accuracy divided by the average value of the set. This is a useful quantity because it is dimensionless and independent of the units in which the measurements were made. 2. A quantity expressing the effect of random errors on the location of one point or feature with respect to another. In particular, (a) an evaluation of the effect of the random errors in points on a map with respect to the graticule, excluding any errors in the graticule or the coordinate system of the graticule; or (b) an evaluation of the effect of random errors in determining the location of one point on a map with respect to another point on the same map.

Repeating Theodolite
Designed so that the sum of successive measurements of an angle can be read directly on the graduated horizontal circle. The vertical axis of rotation is represented physically by two concentric spindles. One of these lets the alidade be rotated independently of the horizontal circle and the other, by a clamp, lets the alidade and the horizontal circle rotate together. Also called a double-center theodolite, reiteration theodolite, engineer's transit, and repeating instrument. The value of the angle is obtained by dividing the total arc passed through (the final reading on the circle, plus an appropriate multiple of 360°) in making the series of measurements by the number of times the angle has been measured. In theory, the repeating theodolite is an instrument capable of giving very precise results; in practice, it does not give results as satisfactory as those obtained with a direction theodolite.

Resection
Determining the location of a point by extending lines of known direction from that point to two other known points. Equivalently, a procedure of determining the location by extending lines making known angles with a third line whose position and length are known. The procedure may be done on the ground using lines of sight as the lines extended, or may be done geometrically by plotting the points and drawing the lines to scale on a sheet, or

may be done algebraically by using the coordinates, angles, etc., of the points and lines involved.

Right Ascension
The angular distance measured eastward on the equator from the vernal equinox to the hour circle through the celestial body, from 0 to 24 hours.

Sea Level Datum of 1929
Adopted as a standard datum for heights. The sea level is subject to some variations from year to year, but, as the permanency of any datum is of prime importance in any engineering work, a sea-level datum after adoption should, in general, be maintained indefinitely even though differing slightly from later determinations of mean sea level based on longer series of observations.

Self-leveling Level
The line of sight is automatically maintained horizontal by means of a built-in pendulum device.

Semimajor Axis
The line from the center of an ellipse to the extremity of the longest diameter; i.e., one of the two longest lines from the center to the ellipse. The term is also used to mean the length of the line.

Semiminor 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.

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.

Setup
1. In general, the situation in which a surveying instrument is in position at a point from which observations are made; e.g., measurements made at the last setup. 2. The actual physical process of placing a leveling instrument over an instrument station. 3. The surveying instrument itself when in position at a point from which observations are made. This last definition may be empty; in any event, its usage with this precise meaning seems to be rare. The terms instrument station and setup, especially in leveling, are often used interchangeably. However, if any distinction is made, setup is considered to be the instrument when mounted or set up over the instrument station or point on the ground which is on the axis of rotation of the instrument. 4. The horizontal distance from the fiducial mark on the front end of a tape, or on that end of the tape which is in use at the time, measured forward to the point, on the mark or monument, to which the particular distance is being measured. A setup is usually very small when measuring a baseline with stakes for supporting the tape put in place before measuring begins. If the distance between stakes is too great, the tape will not reach from stake to stake, and setups must be measured. If portable supports such as bucks are used, there will seldom be need for measuring setups. Setups are positive corrections to taped distances. 5. The positions given those parts of a stereoscopic plotting instrument which are adjustable but remain fixed while the instrument is in use.

Sidereal Day
The interval of time from a transit of the (true) vernal equinox across a given meridian to its next successive transit across the same meridian.

Sidereal Time
Time based upon the rotation of the earth relative to the vernal equinox.

Solar Day
1. The interval of time from the transit of either the sun or the mean sun across a given meridian to the next successive transit of the same body across the same meridian. 2. The duration of one rotation of the sun.

Spheroid
A mathematical figure closely approaching the geoid in form and size. Used as surface of reference for geodetic surveys.

Spirit Level
A closed glass tube (vial) of circular cross section. Its center line forms a circular arc, its interior surface is ground to precise form and filled with ether or liquid of low viscosity, enough free space being left for the formation of 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. Also called the anallactic constant and anallatic constant. It is not a constant for internally focusing telescopes.

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
The algebraic sign and, to some extent, magnitude bear a fixed relation to some condition or set of conditions. In other words, an error which is, in theory at least, predictable and therefore is not a random error. Systematic errors are regular and therefore can be determined a priori. They are usually eliminated from a set of observations before applying the method of least squares to reduce or eliminate random errors. They are classified as theoretical (external) errors, instrumental errors, and personal errors, according to their origin and nature.

State Plane Coordinate System (SPCS)
A reference coordinate system used by the various states of the U.S.

Strength of Figure
A number relating the precision with which lengths of sides in a triangulation network can be determined, to the sizes of the angles, the number of conditions to be satisfied and the distribution of baselines and points of fixed location. Strength of figure, in triangulation, is not based on an absolute scale but rather is an expression of relative precision. The number is really a measure of a network's weakness, because the number increases in size as the strength decreases. By summing the values obtained for the simple figures composing a triangulation network, the strength of figure of the network is obtained. Because a triangulation network is usually composed of several different sets of simple figures, comparable numbers for the different sets can be obtained, and the route giving the strongest total network can then be selected for calculating lengths. Reconnaissance for a proposed triangulation network is usually done under instructions that specify limiting values of the strength of figure for the best and second-best chains of triangles between adjacent baselines; the sites for stations and for baselines are selected accordingly. Where desirable, the length of a section may be reduced by inserting an additional baseline, and the numbers for strength of figure reduced accordingly.

Strip Photography
Using a strip camera. The image of an object moving with respect to the camera is focused onto a surface containing a narrow slit. The film is moved along behind the slit, or the slit is moved over the surface of the film, at a rate compensating for the movement of the object. This method of image-motion compensation is much used in aerial photograph.

Structural Deformation Studies
Not only observations of the changes in the dam structure but the monitoring tries to answer the question of why these occurrences happen.

Subtense Bar
Carrying two easily visible marks a fixed, known distance apart and used for determining the distance from an observer to the bar by means of the angle subtended at the observer by the marks on the bar. The marks are often placed on fittings mounted at the ends of the bar and connected by invar wires under slight but constant tension. This counteracts the effect of temperature on the distance between marks. In use, the subtense bar is mounted horizontally on a tripod and centered over the mark to which distance is to be determined. The angle subtended at the observing instrument is measured and automatically converted by the instrument to distance. The observer reads off the distance, not the subtended angle.

Tangent Plane Grid System
The origin at the point of tangency. Usually the origin is designated 10,000 N and 10,000 E, or some similar amounts, to keep all coordinates positive. This system never extends for any great distance.

Taping
Measuring a distance on the earth, using a surveyor's tape. The persons who mark the two ends in taping are called contact men or, particularly when measuring baselines, tape men or tapemen. The term taping is used in this sense in all surveys except in those of the public lands of the U.S.; in those surveys, for historical and legal reasons, the terms chaining and chainmen are preferred.

Three-wire Leveling
Three horizontal lines are used. 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
Showing the horizontal and vertical locations of the natural and man-made features represented. Also called a relief map. A topographic map is distinguished from a planimetric map by the presence, in the former, of symbols showing relief in measurable form. A topographic

map usually shows the same features as a planimetric map but uses numbered contour lines or comparable symbols to show mountains, valleys, and plains. In the case of hydrographic charts, it uses symbols and numbers to show depths in bodies of water. A topographic map differs from a hypsographic map in that, on the latter, vertical distances are shown with respect to the geoid, while on the former, vertical distances may be referred to any suitable and specified surface.

Transformation
Converting a position from Universal Transverse Mercator (UTM) or other grid coordinates to geodetic, and vice versa; from one datum and ellipsoid to another using datum shift constants and ellipsoid parameters.

Transit
The apparent passage of a star or other celestial body across a defined line of the celestial sphere, as a meridian, prime vertical, or almucantar. The apparent passage of a star or other celestial body across a line in the reticule of a telescope, or some line of sight. The apparent passage of a smaller celestial body across the disk of a larger celestial body. The transit of a star across the meridian occurs at the moment of its culmination, and the two terms are sometimes used as having identical meanings; such usage is not correct, even where the instrument is in perfect adjustment. At the poles, a star may have no culmination but it will transit the meridians.

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. It meets the assumptions on which it is based only when the courses are parallel to the axes of the coordinate system used. Care must be taken in applying the rule because there is not universal agreement on what sign should be used for the misclosure.

Transverse Mercator Projection
The central meridian, or a pair of lines virtually parallel to the central meridian, is mapped into a straight line and exactly to scale. It is essentially the same as a Mercator map-projection preceded by a rotation of the ellipsoid through 90° about a major axis. Equivalently, it is the same as a Mercator map-projection calculated for a cylinder with axis in the equatorial plane.

Traverse
1. A route and the sequence of points on it at which observations or measurements are made.
2. A route, the sequence of points on it at which measurements are made, and the measurements themselves.
3. The process by which a route and a sequence of points of measurement on it are established.

Triangulation
The points whose locations are to be determined, together with a suitable number (at least two) of points of known location, are connected in such a way as to form the vertices of a network of triangles. The angles in the network are measured and the lengths of the sides (i.e., the distances between points) are either measured (at least one distance must be measured or calculated from known points) or are calculated from measured angles and measured or calculated distances. The sides having measured lengths are called baselines. Classically, only a very few short baselines are in the network; these are connected by the sides of triangles of normal size by a sequence of triangles of increasing size. Triangulation permits sites for stations and baselines favorable for use both from topographic and geometric considerations to be selected. It is well adapted to the use of precise instruments and methods and can yield results of great accuracy and precision. It has been used generally where the region to be surveyed was large. The term can be considered as including not only the actual operations of measuring angles and baselines, and the reduction of the data, but also the reconnaissance which precedes those operations and any astronomic observations which are required.

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. Some surveying instruments have a four-armed base or quadribrach in which are the foot screws. Tribrachs are used instead of quadribrachs for control surveys because they do not introduce strains into the base of the instrument; quadribrachs do. Some strains tend to change the instrument's orientation in azimuth during the measuring.

Trigonometric Leveling
The determination of differences of elevation trigonometrically from observed vertical angles and measured or computed horizontal or inclined distances.

Trilateration (positioning)
Method of position determination using the intersection of two or more distances to a point.

Universal Transverse Mercator
A worldwide metric military coordinate system rarely used for civil works applications.

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

U.S. Survey Foot
The unit of length defined by the relationship
1 foot ≡ (1/3)(3,600/3,937) m. Established by the U.S. Coast and Geodetic Survey as published in its Bulletin No. 26 (5 April 1893).

Variance-Covariance Matrix
A matrix whose element in row i and column j is the average value of $(x_i - \mu_i)(x_j - \mu_j)$, where x_i and x_j are random variables with average values μ_i and μ_j, respectively. The elements along the main diagonal are called the variances of the corresponding variables; the elements off the main diagonal are called the covariances.

Variance of Unit Weight
Arbitrary scale factor multiplied by the variance-covariance matrix to obtain the weight matrix.

Vernal Equinox
That point of intersection of the ecliptic and the celestial equator, occupied by the sun as it changes from south to north declination, on or about March 21. Same as first as Aries; first point of Aries; March equinox.

Vernier
An auxiliary scale sliding against and 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. The vernier makes it possible to read a principal scale (such as a divided circle) much closer than one division of that scale. If a division on the primary scale is longer than a single division on the vernier, it is a direct vernier; if a division on the vernier is the longer, it is a retrograde vernier, so called because its numbers run in the opposite direction from those on the primary scale. The direct vernier is the usual type and is used in reading the circles on an engineer's transit and on a repeating theodolite. Two verniers extending and numbered in opposite directions from the same index form a *double vernier*, used in reading a circle having graduations numbered in both directions. A *single vernier* so constructed and numbered that it may be read in either direction is termed a *folding vernier*.

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

Vertical Adjustment Residual
Difference in measured and predicted vertical observations.

Vertical Angle
An angle in a vertical plane. One of the sides of a vertical angle is usually either (a) a horizontal line in the vertical plane or (b) a vertical line in that plane. In case (a), the angle is then called, also, the angular elevation or angular depression or altitude. In case (b), it is also called the zenith angle. The vertical angle between two lines, neither of which is horizontal or vertical, is usually obtained by a combination of two vertical angles as defined previously.

Vertical Circle
1. A great circle of the celestial sphere, through the zenith and nadir. Vertical circles are perpendicular to the horizon. 2. A graduated disk mounted on an instrument in such a manner that the plane of its graduated surface can be placed in a vertical plane. It is primarily used for measuring vertical angles in astronomical and geodetic work.

Vertical Datum
Any level surface (as for example mean sea level) taken as a surface of reference from which to reckon elevations. Although a level surface is not a plane, the vertical datum is frequently referred to as the datum plane. Also called datum level, reference level, reference plane, vertical control datum, vertical geodetic datum.

Vertical Refraction
That component of refraction occurring in a vertical plane. Two kinds of vertical refraction affect surveying: that caused by the curvature of the atmospheric layers of different densities (important principally in aerial photogrammetry), and that caused by temperature gradients near the ground (important principally in spirit leveling and trigonometrical leveling).

Weight
The force with which a planet or satellite attracts a body on its surface. In particular, the force with which the earth attracts a body on its surface. The weight is equal

to the product of the mass of the body by the acceleration imparted by gravity. It should not be confused with the mass of the body (as the U.S. Bureau of Standards does). Note that the body must be on the surface of the planet or satellite or attached to it in some manner. Otherwise the body does not participate in the rotation of the attracting body.

Weighted Mean
1. The sum of the products of each number in a set {X} multiplied by a corresponding number in a set (w), this sum being divided by the sum of the numbers in set (w); i.e., the weighted mean X(W) is given by $(\Sigma wX)/(\Sigma w)$. The quantities w are often defined so that they lie between 0 and +1 and $\Sigma w = 1$. Also called the weighed arithmetic mean. 2. If X is a continuous function with the known probability density p(X), then the weighted mean of X is the integral of [p(X)X]dX.

World Geodetic System of 1984 (WGS 84)
A set of constants specified by the U.S. Department of Defense in 1984 and consisting of the following constants: a datum, the coordinate system of which has its origin at the earth's center of mass; the coordinates, in that datum, of a number of points around the world; constants for transforming from other datums to the System's datum; and values determining the earth's gravity-potential.

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. The wye level was invented in 1740 by J. Sissons. The adjustments made possible by this mounting are peculiar to the instrument. The wye level is one of two general classes of leveling instruments, the other being represented by the dumpy level.

Zenith
The point where an infinite extension of a plumb (vertical) line, at the observer's position, pierces the celestial sphere above the observer's head.

Zenith Angle
Measured in a positive direction downward from the observer's zenith to the object observed. Also called zenith distance and colatitude. It is usually denoted by ζ or by z. The former is preferred except when it may be confused with the deflection of the vertical (also usually denoted by ζ).

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

Appendix C
Guide Specification

GUIDE SPECIFICATION

FOR

DEFORMATION MONITORING AND CONTROL SURVEYING ACTIVITIES

INSTRUCTIONS

1. General. This guide specification is intended for use in preparing Architect-Engineer (A-E) contracts for control survey services. These specifications are applicable to all surveying and mapping contracts used to support US Army Corps of Engineers (USACE) civil works design, construction, operations, maintenance, regulatory, and real estate activities. This guide specification is intended for contracts which are obtained using PL 92-582 (Brooks Act) qualification-based selection procedures.

2. Coverage. This guide specification contains the technical standards and/or references necessary to specify control survey requirements. This guide supports all types of control surveying performed on USACE projects.

3. Applicability. The following types of negotiated A-E contract actions are supported by these instructions:

a. Fixed-price control survey service contracts.

b. Indefinite delivery type (IDT) control survey contracts.

c. A multi-discipline surveying and mapping IDT contract in which control survey services is a line item supporting other surveying, mapping, and/or photogrammetry services.

d. A work order or delivery order placed against an IDT contract.

e. Design and design-construct contracts that include incidental surveying and mapping services (including Title II services). Both fixed-price and IDT contracts are supported by these instructions.

4. Contract Format. The contract format outlined in this guide follows that prescribed in the current edition of the Engineer Federal Acquisition Regulation Supplement (EFARS). The contract format is designed to support PL 92-582 (SF 252) qualification-based A-E procurement actions.

5. General Guide Use. In adapting this guide specification to any project, specific requirements will be changed as necessary for the work contemplated. Changes will be made by deletions or insertions within this format. With appropriate adaptation, this guide form may be tailored for direct input in the Standard Army Automated Contracting System (SAACONS). Clauses and/or provisions shown in this guide will be renumbered during SAACONS input.

6. Insertion of Technical Specifications. Engineer Manual (EM) 1110-1-1004, Deformation Monitoring and Control Surveying should be attached to and made part of any service contract for control surveying. This manual contains complete specifications and quality control criteria for the total (field-to-finish) execution of these surveys.

a. Technical specifications for control surveying that are specific to the project (including items such as the scope of work, procedural requirements, and accuracy requirements) will be placed under Section C of the SF 252 (Block 10). The prescribed format for developing the technical specifications is contained in this guide specification. Project-specific technical specifications will not contain contract administrative functions -- these should be placed in more appropriate sections of the contract.

b. Technical specifications for other survey functions required in a control survey services contract should be developed from other Civil Works Construction Guide Specifications applicable to the surveying and mapping discipline required.

c. Standards and other specifications should be checked for obsolescence and for dates and applicability of amendments and revisions issued subsequent to the publication of this specification. Use Engineer Pamphlet (EP) 25-1-1, Index of USACE/OCE Publications. Maximum use should be made of existing EM's, Technical Manuals, and other recognized industry standards and specifications.

d. Throughout Section C of this guide, the specification writer must elect a contract performance method:

(1) The Government designs the survey occupation/observing schedule.

(2) The contractor designs his performance method based on the criteria given in EM 1110-1-1004.

Selection of the first method depends on the survey expertise of the specification writer. This method also transfers much of the contract risk to the Government. The second method is the preferred contract procedure.

7. Alternate Clauses/Provisions or Options. In order to distinguish between required clauses and optional clauses, required clauses are generally shown in capital letters. Optional or selective clauses, such as would be used in a work order, are generally in lower case. In other instances, alternate clauses/provisions may be indicated by brackets "[ ]" and/or clauses preceded by a single asterick "*". A single asterick signifies that a clause or provision which is inapplicable to the particular section may be omitted, or that a choice of clauses may be made depending upon the technical surveying and mapping requirement. Clauses requiring insertion of descriptive material or additional project-specific specifications are indicated by either ellipsis or underlining in brackets (e.g., "[. . .]" "[____]"). In many instances, explanatory notes are included regarding the selection of alternate clauses or provisions.

8. Notes and Comments. General comments and instructions used in this guide are contained in asterisk blocks. These comments and instructions should be removed from the final contract.

9. Indefinite Delivery Type (IDT) Contracts and Individual Work Order Assignments. Contract clauses which pertain to IDT contracts, or delivery orders thereto, are generally indicated by notes adjacent to the provision. These clauses should be deleted for fixed-price contracts. In general, sections dealing with IDT contracts are supplemented with appropriate comments pertaining to their use. Work orders against a basic IDT contract may be constructed using the format contained in Section C of this guide. This contract section is therefore applicable to any type of surveying service contracting action.

TABLE OF CONTENTS

THE CONTRACT SCHEDULE

SECTION A

SOLICITATION/CONTRACT FORM

****
NOTE: Include here Standard Form 252 in accordance with the instructions in EFARS.
****

SF 252 -- (Block 5): PROJECT TITLE AND LOCATION

****
NOTE: Sample title for fixed-price contract:
****

PRIMARY GEODETIC CONTROL SURVEYS IN SUPPORT OF SITE PLAN MAPPING FOR PRELIMINARY
CONCEPT DESIGN OF FAMILY HOUSING COMPLEX BETA, FORT ___________________, VIRGINIA.

PRIMARY GEODETIC REFERENCE SURVEYS FOR BOUNDARY DEMARCATION SURVEYS OF __________________[PROJECT], ______________________, OHIO.

****
NOTE: Sample title for indefinite delivery type contract:
****

INDEFINITE DELIVERY CONTRACT FOR PROFESSIONAL PRECISE SURVEYING SERVICES IN SUPPORT
OF VARIOUS *[CIVIL WORKS] [MILITARY CONSTRUCTION] PROJECTS *[IN] [ASSIGNED TO] THE
____________________________________ DISTRICT.

SECTION B

SERVICES AND PRICES/COSTS

****
NOTE: The fee schedule for control survey services should be developed in conjunction with the preparation of the independent government estimate (IGE) along with the technical specifications. Procedures for determining unit costs are described in EM 1110-1-1004. Determination of estimated unit costs/prices should conform to the detailed analysis method, or "seven-item breakdown," as described in this manual and other USACE publications. The fee schedule should follow the general unit price example format shown below, which is more fully described in EM 1110-1-1004. Select only those line items applicable to a particular contract or project. A separate fee schedule for each option period should be developed and negotiated during contract negotiations and included with the contract during initial award.

Daily Rate Schedule Example: For IDT contracts, a unit quantity for each line item would normally be negotiated. Actual quantities would be used on a fixed price contract. Daily units of measure (U/M) may be modified to hourly or other nominal units if needed. Prices may also be scheduled on a work unit basis (e.g., linear miles, feet, per section, area, etc.) as described in EM 1110-1-1004.

****

Price per Control Point/Station Schedule Example:

ITEM	DESCRIPTION	QUAN	U/M	U/P	AMOUNT
0001	Horizontal Control Station, 2nd Order, Class I	[1]	Station		
0002	Horizontal Control Station, 2nd Order, Class II	[1]	Station		
0003	Horizontal Control Station, 3rd Order, Class I	[1]	Station		
0004	Horizontal Control Station, 3rd Order, Class II	[1]	Station		
0005	Vertical Control Station, 2rd Order, Class I	[1]	Station		
0006	Vertical Control Station, 2nd Order, Class II	[1]	Station		
0007	Vertical Control Station, 3rd Order	[1]	Station		
00XX					

Daily Rate Schedule Example:

ITEM	DESCRIPTION	QUAN	U/M	U/P	AMOUNT
0001	Registered/Licensed Land Surveyor -- Office	[1]	Day		
0002	Registered/Licensed Land Surveyor -- Field	[1]	Day		
0003	Civil Engineering Technician -- Field Party Supervisor (Multiple Crews)	[1]	Day		
0004	Engineering Technician (Draftsman) -- Office/Field	[1]	Day		
0005	Supervisory Survey Technician (Field)	[1]	Day		
0006	Survey Technician -- Instrument Man/Recorder	[1]	Day		

0007	Surveying Aid -- Rodman/Chainman (Conventional Surveys)	[1]	Day		
0008	[Two][Three][Four][___] - Man GPS Survey Party [___] GPS Receiver(s) [___] Vehicle(s) [___] Computer(s)	[1]	Day		
0009	Additional GPS Receiver	[1]	Day		
0010	[___] - Man Automated (Electronic) Survey Party	[1]	Day		
0011	[___] - Man Conventional Horizontal Control Survey Party	[1]	Day		
0012	[___] - Man Conventional Vertical Control Survey Party	[1]	Day		
0013	Station Monuments [Disk Type] [Construction Materials]	[1]	EA		
0014	Survey Computer (Office)	[1]	Day		
0015	CADD Processing & Plotting	[1]	Day		
0016	Professional Geodesist Computer (Office)	[1]	Day		
0017	Blue Line Prints	[1]	SQ FT		
00XX					

SECTION C

STATEMENT OF WORK

C.1 GENERAL. THE CONTRACTOR, OPERATING AS AN INDEPENDENT CONTRACTOR AND NOT AN AGENT OF THE GOVERNMENT, SHALL PROVIDE ALL LABOR, MATERIAL, AND EQUIPMENT NECESSARY TO PERFORM THE PROFESSIONAL CONTROL SURVEYING *[AND MAPPING WORK] *[FROM TIME TO TIME] DURING THE PERIOD OF SERVICE AS STATED IN SECTION D, IN CONNECTION WITH PERFORMANCE OF *[__________________] SURVEYS *[AND THE PREPARATION OF SUCH MAPS] AS MAY BE REQUIRED FOR *[ADVANCE PLANNING,] [DESIGN,] [AND CONSTRUCTION] [or other function] [ON VARIOUS PROJECTS] *[specify project(s)] . THE CONTRACTOR SHALL FURNISH THE REQUIRED PERSONNEL, EQUIPMENT, INSTRUMENTS, AND TRANSPORTATION AS NECESSARY TO ACCOMPLISH THE REQUIRED SERVICES AND FURNISH TO THE GOVERNMENT REPORTS AND OTHER DATA TOGETHER WITH SUPPORTING MATERIAL DEVELOPED DURING THE FIELD DATA ACQUISITION PROCESS. DURING THE PROSECUTION OF THE WORK, THE CONTRACTOR SHALL PROVIDE ADEQUATE PROFESSIONAL SUPERVISION AND QUALITY CONTROL TO ASSURE THE ACCURACY, QUALITY, COMPLETENESS, AND PROGRESS OF THE WORK.

****

NOTE: The above clause is intended for use on an IDT contract for control survey services (as applicable). It may be used for fixed-price service contract by deleting appropriate IDT language and adding the specific project survey required. This clause is not repeated on individual delivery orders.

****

C.2 LOCATION OF WORK.

****

NOTE: Use the following clause for a fixed-scope contract or individual work order.

****

C.2.1. A *[PROJECT CONDITION] [PLANS AND SPECIFICATIONS] [____] [specify type] CONTROL SURVEY IS REQUIRED AT [_______________] [list project area or areas required]. *[A GENERAL LOCATION MAP OF THE PROJECT AREA IS ATTACHED AT SECTION G OF THIS CONTRACT.]

****

NOTE: Use the following when specifying an indefinite delivery contract for control surveying.

****

C.2.2. CONTROL SURVEYING SERVICES WILL BE PERFORMED IN CONNECTION WITH PROJECTS *[LOCATED IN] [ASSISGNED] TO THE [__________________] DISTRICT. *[THE __________________ DISTRICT INCLUDES THE GEOGRAPHICAL REGIONS WITHIN *[AND COASTAL WATERS] [AND RIVER SYSTEMS] ADJACENT TO:]

*____

[list states, regions, etc.]

****
NOTE: Note also any local points-of-contact, right-of-entry requirements, clearing restrictions, installation security requirements, etc.
****

C.3 TECHNICAL CRITERIA AND STANDARDS.

REFERENCE STANDARDS:

C.3.1. U.S. ARMY CORPS OF ENGINEERS ENGINEER MANUAL (EM) 1110-1-1004, DEFORMATION MONITORING AND CONTROL SURVEYING. THIS REFERENCE IS ATTATCHED TO AND MADE PART OF THIS CONTRACT (SEE SECTION G).

C.3.2. U.S. ARMY CORPS OF ENGINEERS EM 1110-1-1002, SURVEY MARKERS AND MONUMENTATION. *[THIS REFERENCE IS ATTACHED TO AND MADE PART OF THIS CONTRACT. (SEE CONTRACT SECTION G).]

C.3.3. *U.S. ARMY CORPS OF ENGINEERS STANDARDS AND SPECIFICATIONS FOR SURVEYS, MAPS, ENGINEERING DRAWINGS, AND RELATED SPATIAL DATA PRODUCTS *(cite current regulation).

C.3.4. *[Local Drafting Standards containing sheet sizes, types, formats, etc.]

C.3.5. *[List other applicable reference standards which are listed in Appendix A of EM 1110-1-1004 and may be applicable to the project.

****
NOTE: Reference may also be made to other applicable Engineer Manuals, CADD Standards, or other standard criteria documents. Such documents need not be attached to the Contract; if attached, however, reference should be made to their placement in contract Section G.
****

C.4 WORK TO BE PERFORMED. SPECIFIC PROCEDURAL, TECHNICAL, AND QUALITY CONTROL REQUIREMENTS FOR CONTROL SURVEYING *[AND MAPPING SERVICES] TO BE PERFORMED UNDER THIS CONTRACT ARE LISTED IN THE PARAGRAPHS BELOW. UNLESS OTHERWISE INDICATED IN THIS CONTRACT *[OR IN DELIVERY ORDERS THERETO], EACH REQUIRED SERVICE SHALL INCLUDE FIELD-TO-FINISH EFFORT PERFORMED IN ACCORDANCE WITH THE STANDARDS AND SPECIFICATIONS IN EM 1110-1-1004.

****
NOTE: The following clauses throughout paragraph C.4 may be used for either fixed-prices surveying service contracts, IDT work orders under an IDT surveying contract, or IDT where contract survey services are part of a schedule of various survey disciplines. Fixed-scope contracts: Detail specific hydrographic surveying and mapping technical work requirements and performance criteria which are necessary to accomplish the work. IDT contracts and work orders thereof: Since specific project scopes are indefinite at the time a basic contract is prepared, only general technical criteria and standards can be outlined. Project or site-specific criteria will be contained in each delivery order, along with any deviations from technical standards identified in the basic IDT contract. The clauses contained herein are used to develop the general requirements for a basic IDT contract. Susequent delivery orders will reference these clauses; adding project-specific work requirements as required. Delivery order formats should follow the outline established for the basic IDT contract.
****

C.4.1. GENERAL SURVEY REQUIREMENTS. SURVEYS WILL BE PERFORMED USING CONTROL SURVEY METHODS. SUCH METHODS WILL BE MADE IN STRICT ACCORDANCE WITH THE CRITERIA CONTAINED IN EM 1110-1-1004, EXCEPT AS MODIFICED OR AMPLIFIED HEREIN. THE METHOD USED WILL BE *[CARRIER PHASE BASED GPS SURVEY METHODS UTILIZING STATIC *[STOP-AND-GO] *[KINEMATIC] *[PSEUDO-KINEMATIC] TECHNIQUES *[OR COMBINATIONS THEREOF]] *[CONVENTIONAL SURVEY METHODS]. *[CONVENTIONAL SURVEY METHODS WILL BE USED TO DENSIFY SUPPLEMENTAL POINTS RELATIVE TO ESTABLISHED GPS STATIONS.]

****

NOTE: For fixed-scope contracts and/or delivery orders, use the following types of clauses to detail the general work specifications. Terminology would be appropriately modified to cover the type of survey to be done. A general description of the project and any unique purpose of the survey may also be added.

****

C.4.2. HORIZONTAL ACCURACY REQUIREMENTS. NEW *[PRIMARY] STATIONS SHALL BE ESTABLISHED TO A *[_________]-ORDER, *[CLASS *[___]] RELATIVE ACCURACY CLASSIFICATION *[OR ____ PART IN _________]. *[SUPPLEMENTAL POINTS SHALL BE ESTABLISHED TO A *[__________]-ORDER, *[CLASS *[___]] RELATIVE ACCURACY CLASSIFICATION. HORIZONTAL ACCURACY REQUIREMENTS SPECIFIED FOR NEWLY POSITIONED STATIONS SHALL BE BASED ON A FREE (UNCONSTRAINED) ADJUSTMENT OF SURVEY OBSERVATIONS AND SHALL MEET THE RELATIVE ACCURACY AND/OR LOOP MISCLOSURE CRITERIA INDICATED IN EM 1110-1-1004.

****

NOTE: Note that accuracy classifications and related contract quality control and acceptance are based on a free adjustment of the work -- not a constrained adjustment to fixed/existing control. If using GPS to establish primary control, it is often the case that GPS will often give horizontal accuracies exceeding those of the control used to establish the GPS point, especially if the basic control (i.e., reference control) has been established by conventional survey techniques. Horizontal accuracy classifications exceeding Second-Order, Class I (i.e., 1:50,000) will not normally be specified for USACE control work.

****

C.4.3. VERTICAL ACCURACY REQUIREMENTS. NEW *[PRIMARY] STATIONS SHALL BE ESTABLISHED TO A
*[_________]-ORDER, *[CLASS *[___]] RELATIVE ACCURACY CLASSIFICATION *[OR ____ PART IN _________]. *[SUPPLEMENTAL POINTS SHALL BE ESTABLISHED TO A *[__________]-ORDER, *[CLASS *[___]] RELATIVE ACCURACY CLASSIFICATION. VERTICAL ACCURACY REQUIREMENTS SPECIFIED FOR NEWLY POSITIONED STATIONS SHALL BE BASED ON A FREE (UNCONSTRAINED) ADJUSTMENT OF SURVEY OBSERVATIONS AND SHALL MEET THE RELATIVE ACCURACY AND/OR LOOP MISCLOSURE CRITERIA INDICATED IN EM 1110-1-1004.

****

NOTE: When using GPS techniques to establish vertical elevations, extreme caution must be employed -- its application for engineering and contruction work is limited. Refer to EM 1110-1-1003 prior to developing specifications for GPS vertical control densification.

****

C.4.4. PROCEDURAL OBSERVATION REQUIREMENTS. NETWORK DESIGN, STATION OCCUPATION REQUIREMENTS, BASELINE REDUNDANCIES, AND CONNECTION REQUIREMENTS TO EXISTING NETWORKS SHALL FOLLOW THE CRITERIA GIVEN IN EM 1110-1-1004, EXCEPT AS MODIFIED IN THESE SPECIFICATIONS.

****

NOTE: At this point, indicate any exceptions, modifications, and/or deviations from the criteria in EM 1110-1-1004. The specification writer may optionally elect to have the contractor design his observing procedures in accordance with general EM 1110-1-1004 criteria. Alternatively, specific baselines or stations requiring occupation may be specified. Use of either option depends on the surveying experience/expertise of the specification writer. The preferred method is to allow the maximum flexibility for the contractor to determine the most optimum network design (interconnections, traverses, loops, spurs, etc.). In specifying baselines/points that have been monumented, contingencies should be allowed for resetting marks and/or eccentric observations due to obscured satellite visibility.

****

C.4.5. *<u>SPECIFIC BASELINES TO BE MEASURED</u>.

****

NOTE: Use the above clause only if the government specification writer is designing the network.

****

*(1) THE FOLLOWING BASELINES SHALL BE OBSERVED ON THIS PROJECT: [...] *[list specific station-station baselines and any requirements for redundant observations]

*(2) THESE BASELINES ARE INDICATED BY [...] *[specify line symbol] ON THE ATTACHED MAP IN SECTION G.

C.4.6. <u>NEW STATIONS TO BE *[MONUMENTED AND] OCCUPIED</u>.

(1) THE FOLLOWING *[...] *[indicate number of] STATIONS ARE TO BE OCCUPIED AND POSITIONED USING *[indicate survey method] SURVEY TECHNIQUES: *[list/tabulate new station name and/or area designation, accuracy requirements (order/class), redundant occupations, etc.]

(2) THE NEW STATION *[GENERAL LOCATIONS] ARE INDICATED WITH A *[...] *[indicate map symbol used] ON THE ATTACHED MAP. *[ACTUAL STATION LOCATION WITHIN THE GENERALLY DEFINED AREA SHALL BE SELECTED BY THE CONTRACTOR AND SHALL BE LOCATED SUCH THAT ADEQUATE INTERVISIBILITY (AS REQUIRED) IS AFFORDED.]

C.4.7. <u>EXISTING NETWORK CONTROL STATIONS TO BE OCCUPIED AND CONNECTED</u>.

(1) A TOTAL OF *[...] *[specify number of] EXISTING HORIZONTAL CONTROL STATIONS WILL BE USED TO REFERENCE HORIZONTAL OBSERVATIONS ON THIS SURVEY. A LISTING OF THESE FIXED POINTS *[IS SHOWN BELOW] [IS SHOWN IN ATTACHMENT G.*] FIXED COORDINATES ARE *[NAD 27] [NAD 83] [WGS 84 GEOCENTRIC] [...].

****

NOTE: List each existing control station(s) or, alternately, refer to a map or tabulation attachment in contract Section G.

****

(2) A TOTAL OF *[...] [specify number] VERTICAL CONTROL STATIONS (BENCHMARKS) WILL BE OCCUPIED AND USED TO CONTROL AND/OR PROVIDE VERTICAL ORIENTATION REFERENCE TO VERTICAL OBSERVATIONS ON THIS SURVEY. A LISTING OF THESE FIXED BENCHMARKS *[IS SHOWN BELOW] [IS SHOWN IN ATTACHMENT G*]. ELEVATIONS FOR ALL FIXED BENCHMARKS ARE BASED ON *[NGVD 29] [IGLD-55] [NAVD 88] [IGLD-85] [...] DATUM.

****
NOTE: List or reference attachment for existing benchmarks.
****

(3) REQUIRED BASELINE CONNECTIONS TO EXISTING CONTROL ARE SHOWN *[IN SECTION G]. THESE FIXED POINTS WILL BE USED IN PERFORMING A FINAL CONSTRAINED ADJUSTMENT OF ALL NEW WORK. HORIZONTAL POINTS ARE INDICATED BY A [...], VERTICAL POINTS BY A [...], COMBINED POINTS BY A [...], AND NEW BASELINES BY A [...].

****
NOTE: Use the above clause when existing control points to be connected are specified in the contract.
****

(4) ALL HORIZONTAL AND VERTICAL MONUMENTS ARE KNOWN TO BE IN PLACE AS OF *[date]. DESCRIPTIONS FOR EACH POINT *[ARE ATTACHED AT CONTRACT SECTION G]. THE SOURCE AGENCY AND ESTIMATED ACCURACY OF EACH POINT ARE INDICATED ON THE DESCRIPTION. *[IF VISIBILITY BETWEEN POINTS IS OBSCURED FROM AN EXISTING STATION TO ONE TO BE ESTABLISHED BY THIS CONTRACT, THEN A NEW MARK SHALL BE SET AT THE RATE FOR ITEM [___] IN SECTION B.] *[THE CONTRACTOR'S FIELD REPRESENTATIVE SHALL IMMEDIATELY NOTIFY THE GOVERNMENT'S CONTRACTING OFFICER'S REPRESENTATIVE IF EXISTING CONTROL POINTS HAVE BEEN DISTURBED AND/OR CONDITIONS ARE NOT AS INDICATED IN THE FURNISHED DESCRIPTIONS].

****
NOTE: Use the following clause(s) only when network design and observation schedule/sequence will be determined by the contractor.
****

(5) UNLESS OTHERWISE SPECIFIED IN THESE INSTRUCTIONS, AT LEAST *[ONE] [TWO] [THREE] [___] EXISTING (PUBLISHED) CONTROL STATIONS MUST BE OCCUPIED IN THE NETWORK. CONNECTION METHODS AND REDUNDANCY ARE AT THE CONTRACTOR'S OPTION. PRIOR TO USING ANY CONTROL POINTS, THE MONUMENTS SHALL BE CHECKED TO ENSURE THAT THEY HAVE NOT BEEN MOVED OR DISTURBED.

C.4.8. <u>NEW STATION MONUMENTATION, MARKING, AND OTHER CONTROL REQUIREMENTS</u>.

(1) ALL STATIONS SHALL BE MONUMENTED IN ACCORDANCE WITH EM 1110-1-1002, SURVEY MARKERS AND MONUMENTATION. MONUMENTATION FOR THIS PROJECT SHALL BE TYPE *[...] FOR HORIZONTAL AND TYPE *[...] FOR VERTICAL, PER EM 1110-1-1002 CRITERIA. *[MONUMENTATION SHALL BE DEFINED TO INCLUDE THE REQUIRED REFERENCE MARKS AND AZIMUTH MARKS REQUIRED BY EM 1110-1-1004.] *[ALL MONUMENTS FOR NEW STATIONS ARE CURRENTLY IN PLACE AND DESCRIPTIONS ARE ATTACHED AT SECTION G.]

****
NOTE: Deviations from EM 1110-1-1004 should be indicated as required. USACE project control rarely requires supplemental reference/azimuth marks -- the optional specification clauses below should be tailored accordingly.
****

*(2) AT EACH STATION, ANGLE AND DISTANCE MEASUREMENTS SHALL BE MADE BETWEEN A NETWORK STATION AND REFERENCE MARKS AND AZIMUTH MARKS SET WHICH WERE ESTABLISHED IN ACCORDANCE WITH THE REQUIREMENTS SET FORTH IN EM 1110-1-1004. ANGULAR AND DISTANCE MEASUREMENT PROCEDURES FOR REFERENCE/AZIMUTH MARKS

SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS SET FORTH IN FGCS STANDARDS AND SPECIFICATIONS FOR GEODETIC CONTROL NETWORKS. ALL OBSERVATIONS SHALL BE RECORDED IN A STANDARD FIELD BOOK.

*(a) FOR REFERENCE MARKS, TWO (2) DIRECTIONAL POSITIONS ARE REQUIRED (REJECT LIMIT ±10-SECOND ARC) AND WITH STEEL TAPING PERFORMED TO THE NEAREST ±0.01 FOOT.

*(b) FOUR DIRECTIONAL POSITIONS ARE REQUIRED TO AZIMUTH MARKS. THE REJECT LIMIT FOR A 1-SECOND THEODOLITE IS ±5 SECONDS. AZIMUTH MARK LANDMARKS SHALL BE EASILY DEFINED/DESCRIBED NATURAL FEATURES OR STRUCTURES WHICH ARE OF SUFFICIENT DISTANCE TO MAINTAIN A *[± ___]-SECOND ANGULAR ACCURACY. *[_______-ORDER ASTRONOMIC AZIMUTHS SHALL BE OBSERVED TO AZIMUTH MARKS.]

*(c) A COMPASS READING SHALL BE TAKEN AT EACH STATION TO REFERENCE MONUMENTS AND AZIMUTH MARKS.

C.4.9. STATION DESCRIPTION AND RECOVERY REQUIREMENTS.

(1) STATION DESCRIPTIONS AND/OR RECOVERY NOTES SHALL BE WRITTEN IN ACCORDANCE WITH THE INSTRUCTIONS CONTAINED IN EM 1110-1-1002. [FORM *[_______] SHALL BE USED FOR THESE DESCRIPTIONS.] DESCRIPTIONS SHALL BE *[WRITTEN] [TYPED].

(2) DESCRIPTIONS *[ARE] [ARE NOT] REQUIRED FOR *[EXISTING] [AND/OR NEWLY ESTABLISHED] STATIONS.

(3) RECOVERY NOTES *[ARE] [ARE NOT] REQUIRED FOR EXISTING STATIONS.

C.4.10. MINIMUM OCCUPATION TIMES FOR OCCUPIED BASELINES. BASELINES SHALL BE OCCUPIED FOR A PERIOD OF TIME WHICH IS CONSISTENT WITH THE SPECIFIED ACCURACY REQUIREMENT FOR THE PROJECT AND/OR PARTICULAR NEW STATION/LINE. SPECIFIC OCCUPATION TIMES CONSISTENT WITH THE ACCURACY REQUIREMENTS ARE BASED ON THE PROCEDURES TO BE FOLLOWED FOR THE SURVEY, AS DETAILED IN EM 1110-1-1004. UNLESS OTHERWISE STATED IN THESE SPECIFICATIONS, THE CRITERIA SHOWN IN THIS MANUAL SHALL BE FOLLOWED FOR EACH PROJECT AND/OR OBSERVED BASELINE.

C.4.11. TYPE AND NUMBER OF SURVEYING EQUIPMENT TO BE DEPLOYED.

(1) THE CONTRACTING OFFICER RESERVES THE RIGHT TO REQUEST PUBLISHED DOCUMENTATION ON THE ACCURACY/QUALITY OF THE HARDWARE/SOFTWARE USED FOR THIS PROJECT. REPORTS PUBLISHED BY THE FGCS MAY BE REQUIRED. ALL SURVEY EQUIPMENT AND SOFTWARE USED UNDER THIS *[CONTRACT] [ASSIGNMENT] SHALL BE SUBJECT TO REVIEW BY THE CONTRACTING OFFICER. EQUIPMENT SUBJECT TO REVIEW SHALL INCLUDE:

(a) *[SPECIFIC SURVEYING EQUIPMENT. . . EDM, TOTAL STATION, LEVEL, ETC.]

(b) *[SPECIFIC SURVEYING EQUIPMENT]

(. . .)

*(2) A MINIMUM OF [. . .] *[SPECIFIC PIECE(S) OF SURVEYING EQUIPMENT] SHALL BE CONTINUOUSLY AND SIMULTANEOUSLY DEPLOYED DURING THIS *[ASSIGNMENT] [PROJECT].

C.4.12. FIELD OBSERVING RECORDING PROCEDURES.

(1) FIELD LOG *[SHEETS] [FORMS] [NOTES] SHALL BE COMPLETED FOR EACH STATION OF EACH SESSION AND SUBMITTED TO THE GOVERNMENT. MINIMUM DATA TO BE INCLUDED ON THESE FIELD LOG RECORDS ARE DESCRIBED IN CHAPTER 7 OF EM 1110-1-1004.

(2) RAW SURVEY DATA, BASELINE REDUCTION DATA, AND ADJUSTMENT SOLUTIONS SHALL BE RECORDED AND SUBMITTED TO THE GOVERNMENT ON *[_____-INCH FLOPPY DISKS] [A PREAPPROVED MEDIUM].

(3) IT SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR TO ASSURE THAT AMPLE OBSERVATIONS ARE CONDUCTED SO THAT ALL POINTS ARE INTERCONNECTED IN A COMPLETE INTERCONNECTING NETWORK OR TRAVERSE SURVEY AND/OR IN ACCORDANCE WITH THE REQUIRED BASELINE MEASUREMENTS SPECIFIED HEREIN. *[ADEQUATE FIELD COMPUTATIONAL CAPABILITY SHALL EXIST IN ORDER TO VERIFY MISCLOSURES PRIOR TO SITE DEPARTURE.]

C.4.13. BASELINE DATA REDUCTION REQUIREMENTS.

(1) SOFTWARE FOR PROCESSING THE SURVEY DATA SHALL BE SUBJECT TO APPROVAL BY THE CONTRACTING OFFICER. ALL SOFTWARE MUST BE ABLE TO PRODUCE FROM THE RAW SURVEY DATA RELATIVE POSITION COORDINATES *[AND CORRESPONDING VARIANCE-COVARIANCE STATISTICS WHICH IN TURN CAN BE USED AS INPUT TO THE NETWORK ADJUSTMENT PROGRAMS].

****

NOTE: Baseline output statistics are generally specified only when least squares adjustments are required, and then only if the specified adjustment software utilizes such statistics.

****

(2) BASELINE PROCESSING SHALL BE COMPLETED FOR ALL BASELINES AND SELECTED FOR USE IN THE FINAL NETWORK ADJUSTMENT BASED ON AN ANALYSIS OF THE STATISTICAL DATA AND RELATIVE SPATIAL RELATIONSHIPS BETWEEN POINTS. TEST CONSTANTS GIVEN FOR A PARTICULAR SOFTWARE SYSTEM SHALL BE COMPARED TO THE PROCESSED RESULTS AND ANY SUSPECT BASELINE THAT DOES NOT MEET THE CRITERIA SHALL BE REOBSERVED OR NOT INCLUDED IN THE FINAL ADJUSTMENT. BASELINE ACCEPTANCE AND REJECTION CRITERIA ARE CONTAINED IN EM 1110-1-1004.

C.4.14. FINAL ADJUSTMENT REQUIREMENTS. SURVEY TRAVERSE LOOPS AND NETWORKS SHALL BE ADJUSTED AND EVALUATED IN ACCORDANCE WITH THE PROCEDURES AND CRITERIA IN CHAPTER 8 OF EM 1110-1-1004. FINAL VECTOR MISCLOSURES MAY BE PROPORTIONATELY DISTRIBUTED AMONG THE OBSERVED BASELINES USING EITHER APPROXIMATE OR LEAST SQUARES ADJUSTMENT TECHNIQUES DESCRIBED IN CHAPTER 8 OF EM 1110-1-1004.

(1) ADJUSTMENTS ARE NORMALLY PERFORMED USING *[SPCS] [UTM] [GEOGRAPHIC] [GEOCENTRIC] [ELLIPSOID] [OTHER] COORDINATES. TRANSFORMED FINAL ADJUSTED HORIZONTAL DATA SHALL BE EXPRESSED IN *[SPCS] [UTM] [GEOGRAPHIC] [GEOCENTRIC] [ELLIPSOID] [OTHER] COORDINATES, AND SHALL BE REFERENCED TO *[NAD 27] [NAD 83] [PROJECT] DATUM. FINAL COORDINATES SHALL BE TABULATED IN *[METERS] [FEET] [OTHER] TO ONLY *[_____] DECIMAL POINTS OF PRECISION. *[FINAL ADJUSTED VERTICAL DATA SHALL BE SHOWN AS ORTHOMETRIC HEIGHTS ON *[NGVD 29] [NAVD 88] [OTHER] VERTICAL DATUM.

SURVEY DEVELOPED ELEVATIONS SHALL BE ROUNDED TO THE NEAREST *[1/100 TH OF A] [1/10 TH OF A] [1/2 OF A] [OTHER] *[METER] [FOOT].]

(2) AN ADJUSTMENT ANALYSIS SHALL INCLUDE THE FOLLOWING:

(a) TRAVERSE LOOPS SHALL BE ANALYZED RELATIVE TO THE INTERNAL CLOSURE CRITERIA GIVEN IN TABLE 3-1 *[OR 3-2] OF EM 1110-1-1004. INTERNAL ACCEPTABILITY OF THE WORK WILL BE BASED ON THE MAGNITUDE OF THE THREE-DIMENSIONAL VECTOR MISCLOSURES RELATIVE TO THE LOOP LENGTH. SUCH LOOP CLOSURE ANALYSIS WILL BE CONSIDERED THE INTERNAL, MINIMALLY CONSTRAINED, FREE ADJUSTMENT. LOOPS/LINES WITH INTERNAL MISCLOSURE RATIOS IN EXCESS OF THOSE SPECIFIED IN THIS CONTRACT SHALL BE REOBSERVED. MISCLOSURES BETWEEN EXTERNAL FIXED CONTRAL MAY BE DISTRIBUTED USING THE APPROXIMATE DISTRIBUTION METHODS GIVEN IN EM 1110-1-1004. FINAL CONSTRAINED ACCURACY ESTIMATES WILL BE BASED ON RELATIVE MISCLOSURES AT FIXED POINTS.

****

NOTE: The following clauses apply only to rigorous least squares adjustment techniques. Note that EM 1110-1-1004 does not require that a rigorous least squares adjustment be performed for USACE control work. The guide writer must establish the technical requirement for such an adjustment to be performed for USACE control work and modify the clauses in this section accordingly.

****

(b) *WHEN A FREE (OR MINIMALLY CONSTRAINED) LEAST SQUARES ADJUSTMENT IS PERFORMED ON THE BASELINE VECTORS, A CLASSIFICATION BASED ON THIS INTERNAL ADJUSTMENT SHALL BE DERIVED AND EVALUATED AGAINST THE MINIMUM ALLOWABLE STANDARDS SHOWN IN CHAPTER 8 OF EM 1110-1-1004 FOR THE GIVEN REQUIRED ACCURACY. THIS FREE ADJUSTMENT, ALONG WITH AN ANALYSIS OF THE BASELINE REDUCTION DATA FOR THE SURVEY, WILL BE USED IN EVALUATING THE CONTRACTUAL ACCEPTABILITY OF THE OBSERVED NETWORK. STATION *[_____________] SHALL BE HELD FIXED AND ANALYZED RELATIVE TO THE CRITERIA CONTAINED IN CHAPTER 8 OF EM 1110-1-1004. THE VARIANCE OF UNIT WEIGHT FOR THE MINIMALLY CONSTRAINED NETWORK ADJUSTMENT SHALL CONFORM TO THE CRITERIA GIVEN IN CHAPTER 8 OF EM 1110-1-1004. RELATIVE LINE ACCURACIES SHALL BE COMPUTED FOR PAIRS OF POINTS ON THE NETWORK USING STATISTICAL DATA CONTAINED IN THE FREE ADJUSTMENT. THESE RELATIVE LINE ACCURACIES SHALL NOT EXCEED THE REQUIRED ACCURACY CLASSIFICATIONS PRESCIBED FOR THE WORK. STATIONS/BASELINES/ NETWORK AREAS WITH FREE ADJUSTMENT RELATIVE ACCURACIES NOT MEETING THE REQUIRED CRITERIA MUST BE REOBSERVED; IT IS THEREFORE CONTINGENT ON THE CONTRACTOR TO ENSURE THAT MISCLOSURE TOLERANCES ARE CHECKED IN THE FIELD.

(c) *A CONSTRAINED LEAST SQUARES ADJUSTMENT WILL BE PERFORMED HOLDING *[FIXED] [PARTIALLY CONSTRAINED] THE COORDINATES OF THE STATIONS LISTED UNDER THE EXISTING CONTROL CLAUSE IN THIS CONTRACT SECTION. FOR THE PURPOSE OF THESE SPECIFICATIONS, BOTH FULLY CONSTRAINED AND PARTIALLY CONSTRAINED POINTS ARE REFERRED TO AS "FIXED" POINTS. *[IF USING ELLIPSOID BASED VALUES, THE CONSTRAINED LEAST SQUARES ADJUSTMENT SHALL USE MODELS WHICH ACCOUNT FOR THE REFERENCE ELLIPSOID FOR THE REFERENCE CONTROL, THE ORIENTATION AND SCALE DIFFERENCES BETWEEN THE SURVEY AND NETWORK CONTROL DATUMS, GEOID-ELLIPSOID RELATIONSHIPS, AND DISTORTIONS AND/OR RELIABILITY IN THE NETWORK CONTROL.]

****
NOTE: A variety of free and/or constrained adjustment combinations may be specified. Specific stations to be held fixed may have been indicated in a prior contract section or the contractor may be instructed to determine the optimum adjustment, including appropriate weighting for constrained points. When fixed stations are to be partially constrained, then appropriate statistical information must be provided -- either variance-covariance matrices or relative positional accuracy estimates which may be converted into approximate variance-covariance matrices in the constrained adjustment.
****

[1] WHEN DIFFERENT COMBINATIONS OF CONSTRAINED ADJUSTMENTS ARE PERFORMED DUE TO INDICATIONS OF ONE OR MORE FIXED STATIONS CAUSING UNDUE BIASING OF THE DATA, AN ANALYSIS SHALL BE MADE AS TO RECOMMENDED SOLUTION WHICH PROVIDES THE BEST FIT FOR THE NETWORK. ANY FIXED CONTROL POINTS WHICH SHOULD BE READJUSTED (TO ANOMALIES FROM THE ADJUSTMENT(S)) SHOULD BE CLEARLY INDICATED IN A FINAL ANALYSIS RECOMMENDATION.

[2] THE FINAL ADJUSTED HORIZONTAL AND/OR VERTICAL COORDINATE VALUES SHALL BE ASSIGNED AN ACCURACY CLASSIFICATION BASED ON THE ADJUSTMENT RESULTS AND IN ACCORDANCE WITH THE CRITERIA INDICATED IN TABLE 3-1 AND/OR TABLE 3-2 OF EM 1110-1-1004. THESE CLASSIFICATIONS SHALL INCLUDE BOTH THE RESULTANT GEODETIC/CARTESIAN COORDINATES AND THE SURVEY RESULTS. THE FINAL ADJUSTED COORDINATES SHALL STATE THE 95 PERCENT CONFIDENCE REGION OF EACH POINT AND THE (TWO-SIGMA) RELATIVE LINE ACCURACY BETWEEN ALL POINTS IN THE NETWORK.

(3) FINAL ADJUSTED COORDINATE LISTING SHALL BE PROVIDED ON HARD COPY *[AND ON] *[___] [specify] COMPUTER MEDIA].

(4) *A SCALED PLOT SHALL BE SUBMITTED WITH THE ADJUSTMENT REPORT SHOWING THE PROPER LOCATIONS AND DESIGNATIONS OF ALL STATIONS ESTABLISHED.

C.5 <u>SUBMITTAL REQUIREMENTS</u>:

C.5.1. SUBMITTAL SCHEDULE: THE COMPLETED SURVEY REPORT SHALL BE DELIVERED WITHIN *[____ DAYS AFTER NOTICE TO PROCEED IS ISSUED] *[BY calendar date].

****
NOTE: Include a more detailed submittal schedule breakdown if applicable to project.
****

C.5.2. SUBMITTED ITEMS: SUBMITTALS SHALL CONFORM TO THOSE SPECIFIED IN EM 1110-1-1004 *[EXCEPT AS MODIFIED HEREIN].

****
NOTE: Reference EM 1110-1-1004 for type survey submittal requirements. Modify and/or add items as required.
****

C.5.3. PACKING AND MARKING: COMPLETED WORK SHALL BE PACKAGED TO PROTECT THE MATERIALS FROM HANDLING DAMAGE. EACH PACKAGE SHALL CONTAIN A TRANSMITTAL LETTER OR SHIPPING FORM, IN DUPLICATE, LISTING THE MATERIALS BEING TRANSMITTED, BEING PROPERLY NUMBERED, DATED, AND SIGNED. SHIPPING LABELS SHALL BE MARKED AS FOLLOWS:

U.S. ARMY ENGINEER DISTRICT, ________________________________
ATTN: ____
*[include office symbol and name]
CONTRACT NO. ______________________________________
*[DELIVERY ORDER NO. ______________________________]
[STREET/PO BOX] ___________________________________
*[complete local mailing address]

*HAND-CARRIED SUBMISSION SHALL BE PACKAGED AND MARKED AS ABOVE, AND DELIVERED TO THE FOLLOWING OFFICE ADDRESS:

____
*[insert office/room number as required]

****
NOTE: In this section, also reference any automated data submittal requirements for survey observations, if applicable.
****

C.6 PROGRESS SCHEDULES AND WRITTEN REPORTS.

C.6.1. *PREWORK CONFERENCE:
****
NOTE: Detail any requirements for a prework conference after contract award, including requirements for preparing written reports for such conferences.
****

SECTION D

CONTRACT ADMINISTRATION DATA

SECTION E

SPECIAL CONTRACT REQUIREMENTS

SECTION F

CONTRACT CLAUSES

****
NOTE: Detailed guidance for preparing the above contract sections is contained in EFARS.
****

SECTION G

LIST OF ATTACHMENTS

G.1. U.S. ARMY CORPS OF ENGINEERS EM 1110-1-1004, DEFORMATION MONITORING AND CONTROL SURVEYING . THIS REFERENCE IS ATTACHED TO AND MADE PART OF THIS CONTRACT.

****

NOTE: List any other attachments called for in contract Section C or in other contract sections. This includes items such as:

a. Marked-up project sketches/drawings.
b. Station/monument descriptions or recovery notes.
c. Lists of connections to existing network.
d. Lists of fixed (existing) stations to be connected with and adjusted to.
e. Drafting standards.
f. CADD standards.

****

SECTION H

REPRESENTATIONS, CERTIFICATIONS, AND OTHER STATEMENTS OF OFFERERS

SECTION I

INSTRUCTIONS, CONDITIONS, AND NOTICES TO OFFERERS

****

NOTE: See EFARS for guidance in preparing these clauses/provisions.

****

Appendix D
CORPSCON Technical Documentation and Operating Instructions

D-1. General

a. Background. The National Geodetic Survey developed the conversion program NADCON (North American Datum Conversion) to provide consistent results when converting to and from North American Datum of 1983 (NAD 83). The technique used is based on a bi-harmonic equation classically used to model plate deflections. NADCON works exclusively in geographical coordinates (Latitude/Longitude).

(1) The U.S. Army Topographic Engineering Center (TEC) created a more comprehensive program CORPSCON (Corps Convert), which is based on NADCON (NADCON operates within CORPSCON). In addition to transformations between NAD 83 and NAD 27 geographical coordinates, CORPSCON also converts between State Plane Coordinates Systems (SPCS), geographical coordinates, and Universal Transverse Mercator (UTM) coordinates, thus eliminating several steps in the total process of converting between grid coordinates on NAD 27 and grid coordinates on NAD 83. Inputs can be in either geographic, UTM, or SPCS coordinates (SPCS 27 X and Y or SPCS 83 northing and easting). This program can also be used to convert between grid and geographic coordinates on the same datum.

(2) The Federal Geodetic Control Subcommittee (FCGS) has adopted NAD 83 as the official horizontal datum for U.S. surveying and mapping activities performed or financed by the Federal Government. The FGCS also stated that, NADCON will be the standard conversion method for all mathematical transformations between NAD 83 and NAD 27. For further information reference Chapter 4 of this EM.

b. Coverage. The current version performs datum conversions for the continental U.S. (CONUS), including the 200-mile commercial zone (CONUS_EXT), Puerto Rico/U.S. Virgin Islands (PR/USVI), Alaska, and Hawaii. **This program does not provide conversion to High Accuracy Reference Networks (HARN's).**

D-2. Source of Program and Assistance

Copies of CORPSCON can be obtained from the address below. An initial distribution of CORPSCON will be made to FOA surveying and mapping points-of-contact concurrently with this documentation. CORPSCON was designed to be easily used by those with minimal computer experience. The following documentation gives detailed instructions for installing and running CORPSCON. If problems with the program are encountered, contact:

Director
U.S. Army Topographic Engineering Center
ATTN: CETEC-TD-GS, (Reference CORPSCON)
7701 Telegraph Road
Alexandria, Virginia 22310-3864
(703) 355-2766

D-3. Program and Data Files

a. CORPSCON consists of the following program files that must be present to run the program:

CORPSCON.EXE	CORPS27.EXE
CORPS83.EXE	CORPSNAD.EXE
CORPSUTM.EXE	CORPSCON. S27
CORPCON. S83	CORPSCON.HLP

b. The information file, READ.ME, contains information on installing and running CORPSCON. The following data files are required to perform datum conversions in CONUS, HAWAII, and PR/USVI:

CONUS.LAS	CONUS.LOS
PRVI.LAS	PRVL.LOS
HAWAII.LAS	HAWAII.LOS

c. The following data files are required for conversions in Alaska:

ALASKA.LAS	ALASKA.LOS
STLRNC.LAS	STLRNC.LOS
STGEORGE.LAS	STGEORGE.LOS
STPAUL.LAS	STPAUL.LOS

d. FOAs are being sent data files depending on their geographic locations. However, any of the above geographic databases can be obtained upon request from the office listed in paragraph D-2.

D-4. Hardware Requirements

An 80286 (or higher) processor Microsoft Disk Operating System (MS-DOS) based personal computer with a math coprocessor is required to run this program. A hard disk drive is recommended, but not required. Conversions from floppy drives take considerably longer. At least one megabyte of disk space (floppy or hard disk) is needed to accommodate the program files, datum model files, and input/output files; and at least 52 kilobytes of Random Access Memory (RAM) is needed for program execution. The CORPSCON program and data files are mailed on 5.25-inch 1.2 megabyte floppy disks (other media are available from the office listed in paragraph D-2). CORPSCON is compatible with most PC monitors, although color monitors (EGA and VGA) provide the most favorable and easily discernible menu display. If a printer is to be used, it must be interfaced through the parallel port (LPT1 or LPT2) in order to be recognized by the program.

D-5. Software Requirements

a. This program requires MS-DOS, version 3.0 or higher. Terminate and Stay Resident (TSR) programs, such as Side Kick, have in the past interfered with the proper execution of CORPSCON. The result of this interference has been erroneous results; therefore it is recommended that the user halt TSR programs prior to executing CORPSCON. Also, other programs that use DOS shells, such as spreadsheets and word processors, must be completely terminated before running CORPSCON.

b. Your CONFIG.SYS file must contain the statement “FILES=20” thus allocating a sufficient number of open files needed for program execution. Consult your DOS manual for further information on how to check and/or modify your CONFIG.SYS file.

D-6. Installation Procedures

a. *Disk requirements*. The CORPSCON program and data files should be copied to a hard disk, if available, for faster program performance and to ensure ample disk space. If a hard disk is not available, then the program can be run from a high density floppy disk drive (at least 1.2 megabytes capacity). This requires that the CORPSCON program and data files for CONUS and PR/USVI be copied onto a single high density floppy disk. A hard disk is required if the Alaska data files are being used.

b. *Hard disk installation.* The following instructions assume you will install CORPSCON on your C drive in a directory named CORPSCON. If you wish to install it on another drive or another directory, change the drive and/or directory designations as appropriate.

(1) At the C:\> prompt, create the CORPSCON directory by typing:

MD C:\CORPSCON <ENTER>

(2) Change to the CORPSCON directory by typing:

CD C:\CORPSCON <ENTER>

(3) Place the CORPSCON floppy disk in the A drive (or B). Note CORPSCON is on a high-density disk. Be certain that your computer has a high-density drive.

(4) At the C:\CORPSCON> prompt, copy the files into the CORPSCON directory by typing:

C0PY A:*.*<ENTER>

(5) At the C:\CORPSCON> prompt, type:

DIR <ENTER>

to verify that the files listed in paragraph (2) were properly copied.

D-7. Operating Instructions

a. Change to the drive/directory containing the program files. Type CORPSCON and hit the enter key to start the program. A single screen menu will be displayed with three windows, “CORPSCON Main Menu,” “Send Data,” and “Console Window.”

NOTE: An identification message at the top of the menu should read “CORPSCON v3.0.” If the message has a number lower than 3.0 or the menu has “Beta CORPSCON v3.0,” then this is an older version of CORPSCON and it should be disregarded.

b. The “CORPSCON Main Menu” has three basic functions that can be selected by typing the highlighted numeric (1, 2, or 3) which is located by that respective function. The “Input Data Format” specifies the coordinate system in which the original coordinates are in. The “Output Data Format” specifies the coordinate system in which the original coordinates are to be converted. CORPSCON displays a listing of supported coordinate systems when either of these two options in the “CORPSCON Main Menu” is selected. Using the cursor keys (Up/Down) the user can move through the different systems. Hitting the Enter key selects the coordinate system which is blinking. Below is a representation of that menu.

```
+------------------------------------------------------------+
|                                                            |
|  Select the Input Data Format                              |
|                                                            |
|     1 - State Plane, NAD 27                                |
|                                                            |
|     2 - State Plane, NAD 83                                |
|                                                            |
|     3 - UTM Coord., NAD 27                                 |
|                                                            |
|     4 - UTM Coord., NAD 83                                 |
|                                                            |
|     5 - Geographic Coordinates, NAD 27                     |
|                                                            |
|     6 - Geographic Coordinates, NAD 83                     |
|                                                            |
+------------------------------------------------------------+
```

c. If selecting either of the grid coordinate systems, CORPSCON prompts the user for units (U.S. Foot, International Foot, or Meters) and the UTM or state plane zone. To select a state plane zone (in NAD 27 or NAD 83), CORPSCON will display a menu in the console window. Using the Page-Up and Page-Down keys and cursor keys, a user can quickly move through several pages of zones and select an individual zone to be used in the conversion. Selection is made by moving the highlighted cursor over the intended zone and hitting the <Enter> key.

NOTE: Throughout the CORPSCON program, the bottom line of the main screen changes showing the user the commands (i.e. function keys) that are valid.

d. The "Input Data Source" specifies the type or format of input data. As above, the selection of this option displays a menu of supported input data sources. Utilization of the cursor keys combined with the <Enter> key allows the user to select a format. Below is a representation of that menu.

```
+------------------------------------------------------------+
|   Select the Input Source for the Data                     |
|     1 - From  Keyboard (Manual Entry)                      |
|     2 - From Batch File (ASCII Format)                     |
|     3 - From NGS Bluebook (*80*/*81*)                      |
|     4 - From GeoLab File                                   |
|     5 - From Fillnet File                                  |
+------------------------------------------------------------+
```

The "From Keyboard" option prompts the user in the console window for a single point to convert upon start of the conversion. A sample of that display is as follows:

For geographic coordinate entry:

```
Enter Input Coordinate..

1 - Enter Latitude:      00 00 00.00000
2 - Enter Longitude:     000 00 00.00000
3 - Enter Name:

0 - To Accept Above Point

Enter Selection -
```

For plane coordinate entry:

```
Enter Input Coordinate..

1 - Enter Easting (X):   0.000
2 - Enter Northing (Y):  0.000
3 - Enter Name:

0 - To Accept Above Point

Enter Selection -
```

Selections are made by selecting the numeric associated with the entry. For example, to enter the station name type 3, and then start typing in the name.

NOTE: A maximum of twenty characters can be used for the name.

e. After entering the coordinates, the values should be checked before pressing the 0, to be certain that the numbers are correct. Pressing 0 starts the conversion computation. If changes need to be made, type the selection number corresponding to the invalid number and reenter the proper coordinate.

NOTE: Points in the Alaska Aleutians that lie in east longitude, i.e., Shimya, must be entered as a west longitude exceeding 180 degrees. For example, a longitude of 174" 05' 12" East would be entered as 185" 54' 48". Also, west longitude, for all points within the U.S. and its territories, is assumed positive.

f. All other selections in the data source menu will prompt the user for a file upon start of the conversion. The formats of these files vary depending on the type of file. The "From Batch file (ASCII format)" is the most common and general format. These files can be created outside of CORPSCON with other programs, e.g., word processors, spreadsheets, etc., that can make an ASCII file. There are two formats that exist for an ASCII batch file, examples of each are listed below:

For geographic coordinates:

```
;This is a sample file
;Agency:  USATEC
;Project:  Sample File,
;NAD 83 Geographic Coordinates
      Test Site,38 44 0.0,77 8 0.0111
```

For grid coordinates:

```
;This is a sample file
;Agency:  USATEC
;Project:  Sample File,
;NAD 83 State Plane Zone 4501,U.S FOOT
      Test Site,11872752.0,6953015.832
```

Comment lines have a ";" in the first colunm of that line. There is no limit on the number of comment lines a file can have. CORPSCON disregards any line starting with a ";". The actual data in the ASCII file are broken down into three fields separated by commas. The first field contains a twenty-character name. The format of the next two fields depends on whether the data are plane or geographic coordinates. For geographic coordinates, the second field contains the latitude (ϕ) and the third contains the longitude (λ) of the station. The latitude and longitude fields contain three values: degrees, minutes, and seconds. Each value is separated by a space. In order for CORPSCON to function properly, there must be a value for each field (even if zero). For plane coordinates, the second field contains the easting (x) and the third field contains the northing (y).

g. Another file format that CORPSCON can use are Bluebook files with *80* and *81* records. Also, CORPSCON can strip geographic coordinates from GeoLab and Fillnet output files. This capability is based on GeoLab version 1.82s (* .lis files) and Fillnet version 3.00 (*.fop files). Outputs generated from different versions of these software packages may not be recognized by CORPSCON if the vendor has not retained common output formats between version releases.

h. The "Send Data" menu allows the user to specify what output formats will be created. There are four options: "To Console," "To Printer," "To Printer File," and "To Data File." The highlighted letter toggles these options on and off, which are, respectively, C, P, F, and D.

i. The "To Console" option displays the original and converted coordinates in the console window (the third area located on the screen). Using the <Page-Up> and <Page-Down> keys a user can cycle through the points. Below is an example of that option.

```
NAME:        Test Site     Record 1 of 1.
             INPUT         OUTPUT
LAT:  38 44 00.00000       N:  6953015.83296
LON:  077 08 00.00000      E: 11872752.62967
     Convergence DD(A) & Scale Factor (K)
             A:  00 51 10.65989
             K: 0.999950422
```

NOTE: After viewing the coordinates in the console window, hit the <ESC> key to continue. No printing will occur until the user has finished viewing the coordinates in the console window.

j. The "To Printer" option sends the data directly to the printer in a publishable format.

k. The "To Printer File" option sends the same publishable format as above to a user specified file. When selecting this option, CORPSCON will prompt the user for a filename in which to write this format.

l. Using either of the above two output options (to the printer or a printer file), CORPSCON creates a specific format that consists of a file header followed by a listing of the input and output data. The header states the form of the inputs (e.g., NAD 27 state plane) and the outputs (e.g., NAD 83 geographic). The input coordinates are listed under the "Input" column and the output coordinates are similarly listed under the "Output" column. If state plane or UTM values are used for input or output, then the convergence and scale factor are listed under the coordinates. If a datum conversion is made from NAD 27 to NAD 83, or vice versa, then the datum shift, in meters, is shown under the coordinates. If the inputs and outputs for a datum conversion are in state plane coordinates, then the state plane coordinate shift, in feet, is shown after the datum shift.

m. The "To Data File" option sends an abbreviated output format to another file. This is an important change from previous versions of CORPSCON in that only the point name and converted coordinates are written to the data file. The exception to the previous statement is that the first four or five lines in the file are comment lines of the format:

```
;Software:  CORPSCON v3.0, Agency: USATEC
;Project:  Sample File,
;Orig. Coord. on NAD 83 Geographic Coordinates
;Trans.Coord. NAD 83 State Plane Zone 4501, U.S. FOOT
    Test Site,11872752.62967, 6953015.83296
```

When selecting this option, CORPSCON will prompt the user for a file name in which to write this format.

n. CORPSCON has several other options that are briefly explained below.

(1) The <ESC> key is used to exit CORPSCON.

(2) The <F1> key displays a help screen that a user can cycle through by using the <Page-Up> and <Page-Down> keys.

(3) The <F2> key allows the user to change the project title and agency that appears in the printouts.

Printout Titles
Project Name: Sample File
Company Name: USATEC

Pressing the <ESC> key in this menu retains the current name listed.

(4) The <F3> key creates a batch file (which the user is prompted for) while in CORPSCON. Depending on the input format specified (geographic or plane coordinates) the user is prompted with the manual input sequence as described previously. The difference is that after typing 0 to select the point, the user is prompted again for another point. This process continues until the user hits the <ESC> key to close the batch file.

(5) The <F4> key starts the conversion.

D-8. Error Messages

CORPSCON is designed to prompt the user for most cases in which a system or runtime error occurs. These errors are as follows:

a. No math co-processor. A math co-processor (hardware item) must be installed in the computer to run the program.

b. No input file. A file with the name specified was not found; enter the correct file name or create the file before running CORPSCON. Check to be certain that the file and program CORPSCON are in same directory and subdirectory.

c. Printer error. The program is unable to send the output to the printer. Check the following:

-printer connection
-printer turned on
-printer interfaced through the parallel port
-paper in printer
-printer in “on-line” or “ready-to-print” status