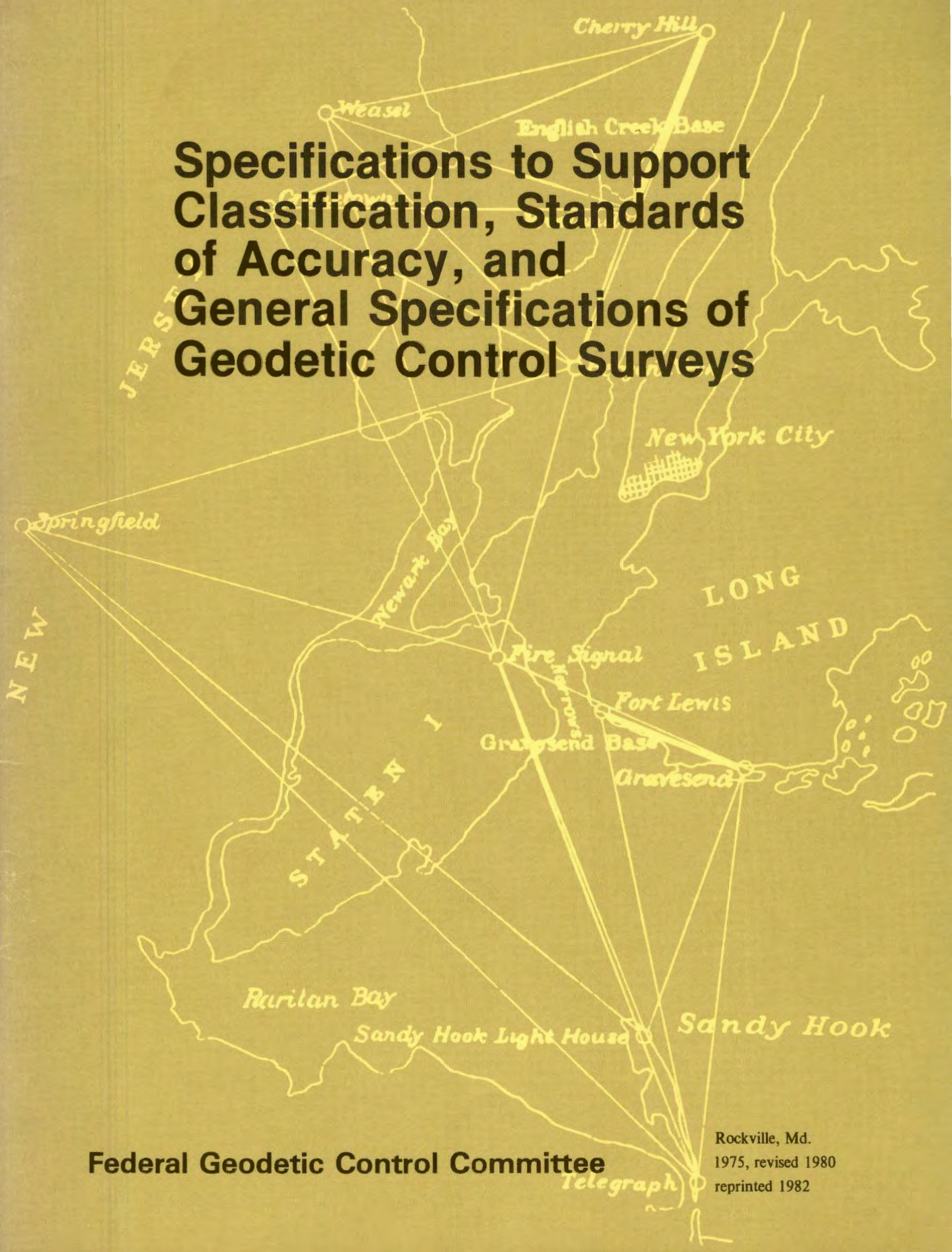
Specifications to Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys



Federal Geodetic Control Committee

Rockville, Md.
1975, revised 1980
reprinted 1982

Federal Geodetic Control Committee

Members:

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Preface

The Office of Management and Budget (OMB), Executive Office of the President, approved the publication of *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* in February 1974. The publication was prepared by the Federal Geodetic Control Committee (FGCC) (1974) and supersedes previous Federal specifications for the establishment of the National Networks of Geodetic Control. All Federal agencies are mandated to comply with these standards in accordance with the provisions of Circular A–16, Revised, titled "Coordination of Surveying and Mapping Activities," promulgated May 6, 1967 (Bureau of the Budget 1967). Specifically: "All surveying activities financed in whole or in part by Federal funds must contribute to the National Networks of Geodetic Control when it is practical and economical to do so." In order to meet the geodetic control needs of Government agencies and the public, all users, both governmental and private, are urged to adhere to these requirements and to cooperate in making the data readily available.

Specifications to Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys was originally published by the FGCC in 1975 to describe and explain the standards of accuracy and to provide guidance in surveying methodology. This current revision clarifies several of the specifications and furnishes additional general information. Added sections discuss the application of recent surveying technology including detailed specifications for determining positions by Doppler satellite methods.

Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys (Federal Geodetic Control Committee 1974), repeatedly referenced herein, is distributed together with this publication to form a set.

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Introduction

To coordinate national mapping, charting, and surveying activities, the Board of Surveys and Maps of the Federal Government was formed December 30, 1919, by Executive Order No. 3206. "Specifications for Horizontal and Vertical Control" were agreed upon by Federal surveying and mapping agencies and approved by the Board on May 9, 1933. When the Board was abolished March 10, 1942, its functions were transferred to the Bureau of the Budget (BOB), now the Office of Management and Budget, by Executive Order No. 9094. The basic survey specifications continued in effect. Bureau of the Budget *Circular* No. A-16, published January 16, 1953 (Bureau of the Budget 1953, 1958), and revised May 6, 1967 (Bureau of the Budget 1967), provides for the coordination of Federal surveying and mapping activities. "Classification and Standards of Accuracy of Geodetic Control Surveys," published March 1, 1957, replaced the 1933 specifications. Exhibit C to *Circular* A-16, dated October 10, 1958 (Bureau of the Budget 1958) established procedures for the required coordination of Federal geodetic and control surveys performed in accordance with the BOB classifications and standards.

The Federal Geodetic Control Committee was chartered December 11, 1968, and a Federal Coordinator for Geodetic Control and Related Surveys was appointed April 4, 1969. The FGCC *Circular* No. 1, "Exchange of Information," dated October 16, 1972, prescribes reporting procedures for the committee (vice Exhibit C of *Circular* A-16) (Federal Geodetic Control Committee 1972).

Rapid evolution of economic and technological factors since 1957 made evident the need for greater flexibility in the design and performance of surveys. As a result, the Federal Geodetic Control Committee formulated *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1974), which provides detailed specifications and general procedural guidance for surveys included in the National Networks of Geodetic Control.

Geodesy, as expressed through control surveys, is a common denominator correlating many of the Nation's and the world's activities in a physical three-dimensional mode. The more commonly used geodetic data include latitudes, longitudes, elevations, azimuths, distances, deflections of the vertical, and gravity values.

One of the many uses for these data is the construction of maps and charts which, in turn, relate the horizontal and vertical positions between all cartographic features on a given map with those features on any other map of the Earth. Thus, the relative accuracy of cartographic presentations is directly related to the accuracy of the National Networks of Geodetic Control.

All of a broad spectrum of maps, charts, and computer-stored data banks require positional accuracy and integrity. These specifications are promulgated for use by both the public and private sectors to aid in ensuring that control surveying activities will be systematically accomplished to meet the Nation's needs.

Control surveys include horizontal, vertical, gravimetric, astronomic, and Doppler satellite surveys. Horizontal control surveys determine geographic positions referenced to a national datum and provide the basis for rectangular coordinate systems. Vertical control surveys determine elevations referred to a national datum derived from mean long-term tidal observations. Astronomic and gravity measurements provide data that contribute to the establishment and adjustment of tbe national control networks.

Surveys of large areas must take into account the curvature of the Earth. For small areas, such as a farm, a city lot, or even a small city, the curvature may be ignored. Larger areas, however, must be surveyed by methods that recognize the mathematical figure of the Earth is a slightly flattened sphere (an ellipsoid of revolution).

Horizontal control is established by triangulation, trilateration, traverse, and Doppler satellite and other extraterrestrial procedures. (In special cases astronomic position determinations or positions from electronic navigation or satellite-based systems may be used.) Triangulation is a system of joined or overlapping triangles in which the length of an occasional side, known as a base line, is measured and the other sides are computed from angles measured at the triangle vertices. Trilateration is a method of surveying in which the lengths of the triangle sides are measured. Traverse is a method in which a sequence of lengths and directions of lines between points is measured to determine the positions of the points.

Doppler satellite surveying can determine geocentric (Earth centered) positions that are accurate to 1 m (meter) or better in each component (x,y,z) by observing a series of passes of one or more satellites of the U.S. Navy Navigation Satellite System (NNSS). Three general methods are used to determine the positions: point positioning which requires the precise (postobservations) ephemeris for the most accurate determinations, the translocation mode which simultaneously uses two or more receivers to track the same satellite, and the short-arc method which also simultaneously tracks the same satellite with two or more receivers. In the translocation mode the satellite orbit is held fixed when the data are reduced, but in the short-arc method the orbit is permitted to take corrections. As a general rule, when employing the multireceiver methods, one receiver remains at the same site until observations are secured at the other locations. A network similar to a net determined by triangulation or trilateration can be developed by using these procedures. Triangulation, trilateration, traverse, and Doppler satellite procedures can be used singly or in combination to obtain the required operational accuracy and precision.

Vertical control is established by spirit or compensator leveling of a high order of accuracy. It consists of the elevations of points with respect to each other and to a common datum. Geodetic leveling follows the geoid and its associated level surfaces, which are irregular, rather than the mathematically determined ellipsoid.

Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys (Federal Geodetic Control Committee 1974) provides the permissible tolerances for the indicated order and class of control. These specifications are used in horizontal control surveys to provide guidance in obtaining the specified strength of figure, spacing of Laplace azimuths, number of observations, triangle closures, side and length checks, and station spacing. For vertical control, the publication provides guidance for satisfying standards for line spacing, field procedures, and section closures. The directives state the manner in which data shall be observed and specify which data shall be assimilated into, or considered to be a part of, the National Networks of Geodetic Control. The Federal Geodetic Control Committee recommends that all other control or precise engineering surveys also adhere to the national specifications and be properly referenced to the national networks.

National Networks of Geodetic Control

The U.S. Department of Commerce's National Oceanic and Atmospheric Administration (NOAA) is responsible for establishing and maintaining the basic national horizontal and vertical geodetic control networks to meet the needs of the Nation. Within NOAA this task is assigned to the National Ocean Survey's Office of National Geodetic Survey (NGS). This responsibility has evolved from legislation dating back to the Act of February 10, 1807 (2 Stat. 413) which created the first scientific Federal agency, known as the "Survey of the Coast." Current authority is contained in United States Code, Title 33, USC 883a, as amended, and specifically defined by Executive Directive, Bureau of the Budget (now the Office of Management and Budget) *Circular* No. A-16, Revised (Bureau of the Budget 1967).

The Federal Coordinator for Geodetic Control and Related Surveys, Department of Commerce, is responsible for coordinating, planning, and executing national geodetic control surveys and related survey activities of Federal agencies, financed in whole or in part by Federal funds. The Executive Directive (Bureau of the Budget 1967: p. 2) states:

> *(1) The geodetic control needs of Government agencies and the public at large are met in the most expeditious and economical manner possible with available resources; and*
>
> *(2) All surveying activities financed in whole or in part by Federal funds contribute to the National Networks of Geodetic Control when it is practicable and economical to do so.*

The Federal Geodetic Control Committee assists and advises the Federal Coordinator for Geodetic Control and Related Surveys.

Accuracy

The standards and specifications for various orders and classes of accuracy are summarized in *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1974: tables 2 and 3). Table 2 of the aforementioned publication refers to horizontal control surveys, in particular those procedures employed for triangulation, trilateration, and traverse. Table 3 of the same publication refers to vertical control surveys. The prescribed standards of accuracy in these two tables pertain only to field observations.

The specifications listed herein are based on the total effort deemed necessary to obtain certain results that are considered the basis for a prescribed standard. Precision is defined as the degree of refinement with which an operation is performed or a measurement stated. The precision of the effort expended contributes largely to the acceptance or rejection of the results. Precision involves, among other considerations, the quality of instruments employed, methods used, repeatability of measurements, and the ability and experience of the personnel. Accuracy is the degree of conformity to a "true" standard or the degree of perfection obtained hy the survey.

Although an absolute guarantee cannot be given that a particular standard will be met if all stated specifications are followed, it is reasonably certain that the closures in length and position will be about one-half of those stated for a particular standard. One of the major factors responsible for survey inaccuracy is undetected or uncorrected atmospheric anomalies. Regardless of rigid adherence to the specifieations, these anomalies occasionally remain undiscovered until future surveys are extended from the stations involved. It is possible that such a discrepancy may not be discovered even after a careful and extensive evaluation of the observational data.

The published accuracies for primary and principal stations of the National Networks of Geodetic Control are derived by comprehensive reviews of the specifications employed, the geometric design of the survey, and statistical evaluations determined from the mathematical adjustment of the survey. These accuracies are in relation to neighboring stations and take into consideration whether or not the points are directly connected. The text of this document uses the terms *accuracy* and *uncertainty* interchangeably and, whether stated or unstated, the terms refer to the relationship of adjacent or neighboring stations. Regardless of terminology, the quantities listed are considered to be at the 2-sigma (2σ) or 95-percent confidence level.

Horizontal Control Surveys

In general, the density and accuracy of control points are directly related to land values and local needs. Table 1 summarizes the classification and standards of accuracy for the various orders and classes of control points in the National Horizontal Control Network. The National Horizontal Control Network must be of such accuracy and precision so that everywhere on the North American Continent, control (both in density and accuracy) meets user or national interest. *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1974) was formulated with this concept in mind. In addition to the observations, other horizontal control information must be recorded to incorporate surveys into the national network. This information is published in *NOAA Manual* NOS NGS 2, Input formats and specifications of the National Geodetic Survey data base, volume I: horizontal control data (Pfeifer 1980).

Nationwide High-Precision Traverses

The high-precision traverses provide scale for the worldwide satellite triangulation network and upgrade the scale and orientation of the National Horizontal Control Network. The traverses consist of a series of high-precision length, angle, and azimuth determinations coincident with geoidal profiles. Their orientation is approximately east-west and north-south, covering the conterminous United States in somewhat rectangular loops. Smaller loops and spur traverses are added to incorporate satellite triangulation stations and areas of special interest.

This publication does not provide eomplete specifieations for high-precision traverses. The following procedural outline indicates the care required to obtain approximately 1 part in 1,000,000 accuracy. This order of survey utilizes methodology and instrumentation necessary to obtain the highest accuracy possible within the state of the art.

As specified originally in 1961, the basic figure was an elongated four-sided polygon, the long sides being nearly parallel. (See fig. 1) The polygons were connected at the ends of the elongated figures, which created a continuous line or chain. Each polygon consisted of two slim triangles created by connecting the two nearby midpoint stations. These midpoint stations were selected to ensure significant differences between the length of the long sides and to maintain sufficient strength of figure, thereby permitting a good mathematical check between the measured lengths. The midpoint stations were usually located within 30 to 50 m of one another and generally

TABLE 1.—*Synopsis of horizontal control classifications*

Attributes	Super first-order	First-order	Second-order class I	Second-order class II	Third-order class I, II
	Orders of surveys and classes of accuracy				
General title	Transcontinental control.	Primary horizontal control.	Secondary horizontal control.	Supplemental horizontal control.	Local horizontal control.
Purpose	Transcontinental traverses. Satellite observations. Lunar ranging. Interferometric surveying.	Primary arcs. Metropolitan area surveys. Engineering projects.	Area control. Detailed surveys in areas of very high land value.	Area control. Detailed surveys in areas of high land value.	Area control. Detailed surveys in areas of moderate and low land value.
Network design	Control develops the national network		Control strengthens the national network.	Control contributes to the national network.	Control referenced to the national framework.
Accuracy	1:1,000,000	1:100,000	1:50,000	1:20,000	1:10,000 1:5,000
Spacing	Traverses at 750 km. Spacing—stations at 15 to 30 km or greater. Satellite as required.	Arcs not in excess of 100 km. Stations at 15 km. Metropolitan area control 3-8 km.	Stations at 10 km. Metropolitan area control at 1-2 km	As required	As required.
Examples of use	Positioning and orientation of North American Continent. Continental drift and spreading studies.	Surveys required for primary framework. Crustal movement. Primary metropolitan area control.	Metropolitan area densification. Land subdivision. Basic framework for densification.	Mapping and charting. Land subdivision. Construction.	Local control. Local improvements and developments.

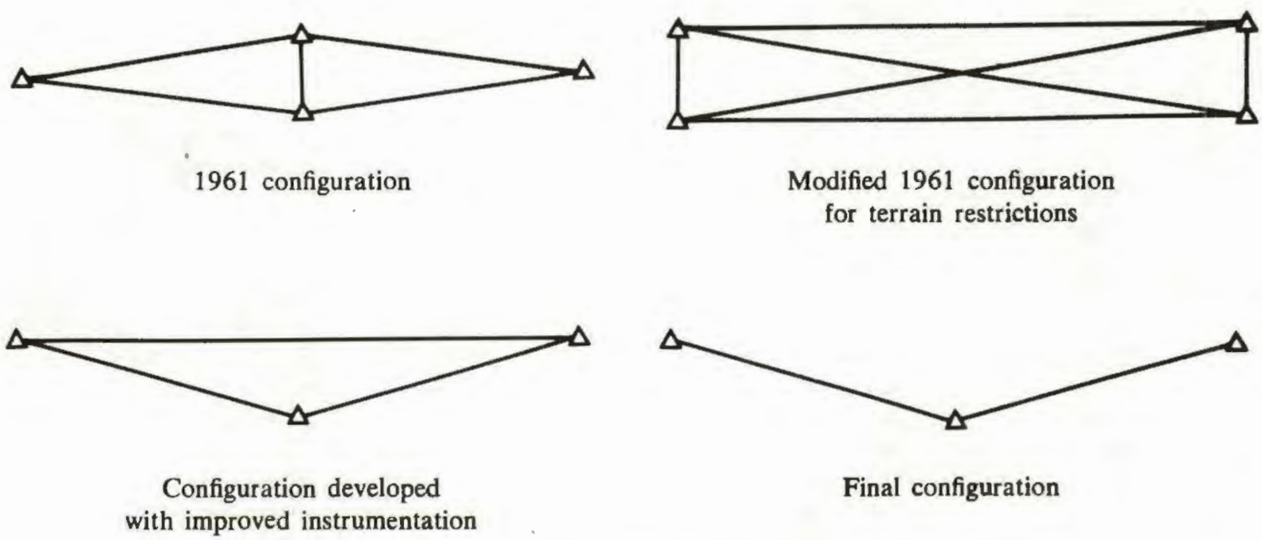

FIGURE 1.—Configurations used for nationwide high-precision traverses.

consisted of a standard station monument and monumented auxiliary point.

Horizontal angles, including "closing the horizon," were measured to first-order specifications on at least two nights by different observers. If the means of the two sets of angles differed by more than 1" (second of arc), a third set was taken on a third night. The average closure of the polygons could not exceed 0."7 and the maximum could not exceed 2."0. In general, the closures in seconds of the slim triangles were not allowed to exceed 600 divided by the short length in meters. The short distances between the midpoint stations were measured using base line taping procedures.

Reciprocal vertical angles were observed over all traverse lines, with ties to bench marks required every four to five polygons. Simultaneous reciprocal observations were required where elevation differences exceeded 100 m. Where differences in station elevations exceeded 500 m, simultaneous reciprocal observations were required both before and after the length measurement of each line. These observations were secured to correct the length observations for variations in atmospheric conditions.

Astronomic position and azimuth observations were required at the connecting stations of the polygons. If the distance between the connecting stations exceeded 35 km, astronomic position and azimuth determinations were required at one of the midpoint stations. First-order and modified first-order (one night's observations) astronomic positions were specified. For azimuth determination, observed directions to all adjacent stations were generally included with the direction to Polaris. Azimuth observations were taken on two nights with a different observer and instrument each night. The average difference between two nights' observations was not allowed to exceed 2."5 and the combined standard error for the two nights could not exceed ±0.45. If either limit was exceeded, a third night's observation was required, with two observers using different instruments and taking successive sets of 16 positions.

Distance measurements were made with electro-optical distance measuring instruments. At least two instruments were used over each line with measurements taken on different nights. Instrument frequencies were checked each week and the recorded results became part of the observation record for that week.

The offset-mirror procedure was introduced in 1962. This technique involved making one or two measurements with the mirror centered over the point and two measurements offset ±0.4 m on the line. The check between the mean distances measured on different nights could not exceed 17 mm plus 1 part per million (ppm). Distances projected through the slim triangles had to agree within 25 mm of the measured lengths of the opposite sides. Meteorological observations consisted of temperature, barometric (altimeter), and humidity readings at instrument height, with midline temperatures at line height obtained using a remote reading thermometer supported by balloon. Angle and distance observations were made at least 10 m above the ground to avoid erratic temperature variations. Observers were alerted to the fact that unfavorable refraction conditions could exist on calm nights necessitating extreme care in operating the equipment.

Maximum care was also exercised in collimating towers and measuring eccentricities. When the wind velocity or direction changed during the night's observations, the tower was recollimated and the eccentricity remeasured.

With the adaptation of laser light sources to electro-optical distance measuring instruments, the range of the instruments was significantly increased. The results of technical improvement and thorough evaluation of data secured over many years enabled several modifications to be made to the original specifications. The original design was primarily developed to ensure that blunders and ambiguities in the observations would be uncovered and resolved. Advanced instrumentation, observing techniques, and procedures have almost completely eliminated the possibilities of blunders.

When it was possible to measure the distance between connecting stations, only one midpoint station was required. The smaller angles of the resulting slender triangle could seldom be larger than 5° and could never exceed 10°. The closure could not exceed 1."7. The check between the measured long side and the projected value derived from the short sides could not exceed 17 mm plus 1 part per million of the longest side.

Further modifications were made as the project progressed. Distance measurements were required on only one night. However, the sequence of observations was such that the distances involved in the projections were measured on different nights employing different instruments. When the figure consisted of a single slender triangle, the short sides were measured on one night and the long side on another night with a different instrument. Also, midline temperature requirements were deleted. Temperature observations were required only at the end points if readings were taken at the minimum 10-meter elevation above ground level and the line of sight cleared all ground elevations by 10 m.

Continuing studies showed conclusively that when proper design was utilized and extreme care exercised, single line traverses would approach the accuracy and internal consistency of past configurations. The network was completed by using this form of traverse. All observations were made during at least two nights with different observers and instruments. Astronomic positions and azimuths were determined at each station, with intermediate astronomic determinations required when the length of line exceeded 35 km. The various tolerances specified for these observations remained unchanged.

The specifications given here for the nationwide high-precision traverses are those which were used to carry out the surveys and may not be considered the best methods or practices today. (Examples include the observing sequences and the allowable tolerances for astronomic azimuths.) Today, more accurate determinations can be obtained by using the procedures suggested for observing astronomic azimuths listed under Astronomic Azimuths (p. 17).

First-Order (Primary Horizontal Control)

The primary horizontal control of the national network consists of monumented stations established with a positional accuracy specification so that the uncertainty between adjacent points should not exceed 1 part in 100,000. Higher accuracy specifications, such as those for the nationwide high-precision traverses, were described previously (p. 3), and may be used in lieu of these specifications for special circumstances.

Accuracy requirements for special projects such as crustal motion studies, high-precision engineering projects, and the testing of defense or space equipment should be at least 1 part in 100,000 and, in some instances, even 1 or 2 parts in 1,000,000.

Second-Order, Class I (Secondary Horizontal Control)

This class consists of monumented stations established with a positional accuracy specification such that the uncertainties between adjacent points should not exceed 1 part in 50,000. This control, established between areas bounded by the high-precision traverses or the primary horizontal control network, densifies the network, thus providing a more advantageous spacing for local use.

In metropolitan areas, when a requirement exists for additional control points more closely spaced than those established under first-order specifications, the additional points should be established by second-order, class I procedures.

Second-Order, Class II (Supplemental Horizontal Control)

This class of control is established with positional accuracy specifications so the uncertainty between adjacent points should not exceed 1 part in 20,000. The demands for reliable horizontal control surveys in areas that are not in a high state of development, or where such activity is not anticipated in the near future, justify the need for the second-order, class II classification. Although points established for the network should be permanently monumented, circumstances can arise when, for various reasons, a few points marked in a temporary fashion and interspersed within the survey system may be justified.

Metropolitan area surveys usually require a second breakdown for control points. These are more closely spaced than control points established to second-order, class I specifications. The second breakdown can be performed at class II standards. Depending on line length and other considerations, traverse specifications relating to the number of angle measurements and azimuth closures may be slightly relaxed. Specifically, modifications that lessen the total effort by as much as one-half may be justified when traverse courses do not exceed 800 m and the number of angle points between higher order control do not exceed 10. *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1974: table 2), lists the recommended number of observations and azimuth closure tolerances for these cases.

Third-Order, Class I and Class II (Local Horizontal Control)

Surveys of this order are established to specifications using methods expected to provide relative accuracies between stations of not less than 1 part in 10,000 and 1 part in 5,000, respectively. Such control nets should be used only to support surveys of lower accuracy within areas of limited extent.

Third-order surveys should be permanently marked, adequately described, and carefully connected to the national network. Spires, stacks, standpipes, flagpoles, and other identifiable objects located to this accuracy have significant value for many surveying and mapping projects. It is especially important that these objects be accurately located and identified if they are visible from navigable bodies of water.

Specifications for surveys less accurate than third order are not included in this publication.

Triangulation

A widely used method for establishing horizontal control is triangulation. This procedure determines the lengths of the sides of a system of joined or overlapping triangles by measuring occasional side lengths and computing the others from angles measured at the vertices. *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1974; table 2) lists by order and class three specifications pertaining to horizontal directions.

Horizontal Directions

The first specification for horizontal directions is a requirement that the instruments must be of specified quality. First-order instruments are defined as theodolites with a direct reading of less than 1″. The Kern DKM-3 and Wild T-3 are representative of this quality. Second-order instruments can be theodolites with direct readings of 1″, for example, the Askania A2, Kern DKM-2, Wild T-2, Zeiss Th-2, etc. A high-quality surveyor's transit or a repeating theodolite is acceptable for third-order surveys. Neither instrument is recommended, however, because of the extra effort required to obtain specified accuracies.

Table 2 specifies the suggested number of observations for various types of transits. The second and third specifications relate to the number of positions to be observed and to the rejection limits. These criteria, derived from experience and statistical evaluations, are based on the accuracy to be obtained, the instrument used, and to some extent the length of the lines involved. The requirements specify that the micrometers must be brought into coincidence twice and both readings recorded for all theodolites. The mean is then used to determine the direction. The difference between the readings for the older models of the Wild T-3 must not exceed 0.3 unit. For the later versions of the Wild T-3 and the Kern DKM-3, this difference cannot be greater than 0.″5. The tolerance for theodolites having a least reading of 1″ (Wild T-2, Kern DKM-2, etc.) cannot exceed 3″. If the tolerances are exceeded, the procedure must be repeated until acceptable values are obtained. There is a further requirement that the initial pointing be verified after setting the circle. Degrees and minutes must be read and recorded for each observation. The practice of reading and recording only the seconds for specific portions is not acceptable. These observing procedures and tolerances also apply to measuring vertical angles. Specifications for first-order and second-order, class I surveys must always be satisfied. In metropolitan areas where

TABLE 2.—*Suggested number of observations using transit or repeating-type instruments*
(The circle settings are given in table 3.)

Accuracy class	Transit*	Number of direct and reverse (D&R) observations per set**	Number of sets	Spread between D&R and sets not to exceed
Third-order class I				
Triangulation	10″	6	2-3	4″
	20	6	4-5	5
	30	6	6-8	6
Traverse	10″	6	1-2	5″
	20	6	2-3	6
	30	6	3-4	7
Third-order class II				
Triangulation	10″	6	1-2	5″
	20	6	2-3	6
	30	6	3-4	7
Traverse***	10″	2	1	5″
	20	4	1	6
	30	6	1	7

*When optical transits reading to 0.1 minute are employed, the "Number of Observations" may be reduced from 6 D&R to 4 D&R. For third-order, class II traverse, 2 D&R continues to be recommended. The "Number of Sets" and "Spread Between D&R and Sets Not to Exceed" remain unchanged. Use appropriate circle settings from table 3.

**A set of observations consists of 6 (or otherwise specified) repetitions of the angle with the telescope in the direct (or reversed) position, followed by 6 (or otherwise specified) repetitions of the explement of the angle in the reverse (or direct) position.

***The specifications given here are for traverses containing fewer than 10 courses between higher-order points. For traverses containing more courses between higher-order control, the "Number of sets" in each case should be 2.

TABLE 3.—*Circle settings*

Two positions of circle				*10-minute micrometer drum*					
1				0°	00′	10″			
2				90	05	40			
	5-minute micrometer drum			*10-minute micrometer drum*			*Wild T-3**		
Four positions of circle							*Circle*		*Micrometer*
1	0°	00′	40″	0°	00′	10″	0°	00′	15″
2	45	01	50	45	02	40	45	02	45
3	90	03	10	90	05	10	90	04	15
4	135	04	20	135	07	40	135	20	45
Six positions of circle									
1	0°	00′	10″	0°	00′	10″	0°	00′	15″
2	30	01	50	30	01	50	30	02	35
3	60	03	30	60	03	30	60	00	50
4	90	00	10	90	05	10	90	04	15
5	120	01	50	120	06	50	120	00	35
6	150	03	30	150	08	30	150	20	50
Eight positions of circle									
1	0°	00′	40″	0°	00′	10″	0°	00′	10″
2	22	01	50	22	01	25	22	00	25
3	45	03	10	45	02	40	45	02	35
4	67	04	20	67	03	55	67	00	50
5	90	00	40	90	05	10	90	04	10
6	112	01	50	112	06	25	112	00	25
7	135	03	10	135	07	40	135	20	35
8	157	04	20	157	08	55	157	00	50
Twelve positions of circle									
1	0°	00′	40″	0°	00′	10″	0°	00′	10″
2	15	01	50	15	01	50	15	00	25
3	30	03	10	30	03	30	30	02	35
4	45	04	20	45	05	10	45	00	50
5	60	00	40	60	06	50	60	00	10
6	75	01	50	75	08	30	75	00	25
7	90	03	10	90	00	10	90	04	35
8	105	04	20	105	01	50	105	00	50
9	120	00	40	120	03	30	120	00	10
10	135	01	50	135	05	10	135	00	25
11	150	03	10	150	06	50	150	20	35
12	165	04	20	165	08	30	165	00	50
Sixteen positions of circle									
1	0°	00′	40″	0°	00′	10″	0°	00′	0″
2	11	01	50	11	01	25	11	00	25
3	22	03	10	22	02	40	22	00	35
4	33	04	20	33	03	55	33	00	50
5	45	00	40	45	05	10	45	02	10
6	56	01	50	56	06	25	56	00	25
7	67	03	10	67	07	40	67	00	35
8	78	04	20	78	08	55	78	00	50
9	90	00	40	90	00	10	90	04	10
10	101	01	50	101	01	25	101	00	25
11	112	03	10	112	02	40	112	00	35
12	123	04	20	123	03	55	123	00	50
13	135	00	40	135	05	10	135	20	10
14	146	01	50	146	06	25	146	00	25
15	157	03	10	157	07	40	157	00	35
16	168	04	20	168	08	55	168	00	50

TABLE 3 (continued)

Transit and repeating type instruments

Sets	*Instrument*** *10″ setting*			*Instrument**** *20″ setting*			*Instrument**** *30″ setting*		
1	0°	00′	00″	0°	00′	00″	0°	00′	00″
2	90	05	30	90	10	20	90	10	30
1	0	00	00	0	00	00	0	00	00
2	60	03	30	60	06	20	60	06	30
3	120	07	00	120	13	00	120	13	00
1				0	00	00	0	00	00
2				45	05	20	45	05	30
3				90	10	00	90	10	00
4				135	15	20	135	15	30
1				0	00	00	0	00	00
2				36	04	20	36	04	30
3				72	08	00	72	08	00
4				108	12	20	108	12	30
5				144	16	00	144	16	00
1							0	00	00
2							30	03	30
3							60	07	00
4							90	10	30
5							120	14	00
6							150	17	30
1							0	00	00
2							25	02	30
3							51	05	30
4							76	08	00
5							102	10	30
6							128	14	30
7							153	17	00
1							0°	00′	00″
2							22	02	30
3							45	05	00
4							67	07	30
5							90	10	00
6							112	12	30
7							135	15	00
8							157	17	30

The Kern DKM-3 theodolite is an example of an instrument with a micrometer drum range of 5′.
The Wild T-2 and the Kern DKM-2 theodolites are examples of instruments with a micrometer drum range of 10′.
* For Wild T-3 theodolities, with a 2′ micrometer graduated to 0.″2, the micrometer readings shown in the table as units would be in seconds.
** Repeating theodolite with a 10′ circle.
*** Transit with a 20′ circle.

traverse courses may be 400 m or less, the number of observations can be reduced when 1″ instruments are employed in the following manner: 8 positions for second-order class I, and 4 positions for second-order class II surveys. If theodolites reading to 0.″2 are used, the number of observations can be reduced to 6 and 4 positions respectively, with a 5″ rejection limit.

For second-order, class II surveys in other than metropolitan areas (or for lower-order surveys), in particular, for traverses containing very short courses, the number of positions and rejection limits can be modified to fit the circumstances. Whenever there is any doubt whether the rejection limits and number of positions meet the specifications, contact the National Geodetic Survey, National Ocean Survey, NOAA, Rockville, MD 20852.

The observations should be made using procedures that minimize collimation, circle, and micrometer errors. By employing the circle settings listed in table 3, most instrument errors will be minimized.

When high precision is required, special care must be exercised in observing horizontal angles over inclined lines. The level of the instrument must be carefully maintained for lines that incline as much as 2°. For lines inclined more than 5°, it is desirable to record striding-level or plate-level readings, using the procedures established for astronomic observations (p. 17), and then correct for the directions involved.

It is important for observers to center the instrument carefully. Measures should be taken to protect the instrument from the heating effect of the Sun and from vibrations caused by wind. Centering and pointing the instrument should be performed with utmost care to eliminate phase error in the target. Phase error is caused by light illuminating the target in an oblique manner which influences the observer to miscenter. Careful pointings should be made if the edges of the target can be determined, probably using double vertical cross hairs rather than the single one. If possible, it is advisable to wait for better observing conditions. The overriding concern should be the stability of the instrument. In addition, the scheduling of observations should take into account those situations in which adverse horizontal refraction is likely to be present. In these cases, the observations should be made during a period when atmospheric conditions will likely permit optimum results.

Triangle Closures

Triangle closures are the simplest test available in the field to ascertain the accuracy of triangulation observations. However, this is only one of two tests that can be made and perhaps the least reliable for evaluating network accuracies. The side check is the second test and probably furnishes the best overall analysis. Triangle closure tolerances specified for first- and second-order triangulation should seldom be exceeded. For first-order triangulation, it is generally expected that the average triangle closure will be about 0.″8, and the maximum closure 2.″5.

Side Checks or Side Equation Tests

In addition to meeting specifications for average and maximum triangle closures, the lengths of the common sides of triangles in the figures, as computed through various chains, must agree within specified limits. For a detailed explanation of side equation tests, see Coast and Geodetic Survey *Special Publication 247*, Manual of geodetic triangulation (Gossett 1959: 158–333). To obtain a side check for a regular quadrilateral in which the triangles were closed by applying one-third of the misclosure to each angle, the ratio of the length of a common side, as computed through the R_1 and R_2 chains, should not differ from one by more than $2.105\ 10^{-6}$ cot α, where α is the smallest angle involved in the computation, multiplied by the following factor for a particular standard:

First-order accuracy	Second-order accuracy Class I	Class II	Third-order accuracy Class I	Class II
1.5	1.5–2	2–4	4	10–12

As an example, let $\alpha = 25°$ and assume a first-order chain; then the ratio should be within $6.76\ 10^{-6}$.

In first-order triangulation involving figures other than regular quadrilaterals, a comparable limiting value for side checks can be obtained through the following formula:

$$0.4 \times 2.105\ 10^{-6} \cot \alpha \times \Sigma\, \Delta.$$

$\Sigma\, \Delta$ is the total number of triangles involved. For triangulation involving second-order, class I and II, and third-order, class I and II, the constant factor in the equation is changed from 0.4 (for first-order specifications) to 0.5, 0.8, 1.0 and 3.0, respectively, for each subsequent order and class.

Side equation tests may also be applied and are recommended as a standard practice. The average correction to an observed direction in seconds of arc is obtained from:

$$T = \frac{\frac{1}{2} S\, \rho''}{\Sigma\, |\cot \alpha| + \Sigma\, |\cot \beta|}$$

where

$$S = \left(\frac{\Pi \sin \alpha}{\Pi \sin \beta} - 1\right)$$

and ρ'' = the number of seconds per radian = $2.06265 \ldots 10^5$. The result should not be greater than the value shown in *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1974: table 2).

To illustrate the procedure, we follow the example given in U.S. Coast and Geodetic Survey *Special Publication* 28, Application of the theory of least squares to the adjustment of triangulation (Adams 1915: 17). We have:

α	$\sin\alpha$	$\lvert\cot\alpha\rvert$	β	$\sin\beta$	$\lvert\cot\beta\rvert$
61°47′35.″0	0.88124617	0.536	26°40′23.″5	0.44890099	1.991
20 50 56.7	0.35590766	2.626	133 53 46.3	0.72059717	0.962
32 09 01.2	0.53214256	1.591	31 03 42.5	0.51596241	1.660
Product of sines $\Pi \sin\alpha =$	0.16690240		$\Pi \sin\beta =$	0.16690186	
Sum of $\lvert\cot\rvert$	$\Sigma \lvert\cot\alpha\rvert =$	4.753		$\Sigma \lvert\cot\beta\rvert =$	4.613

$$S = \left(\frac{\Pi \sin\alpha}{\Pi \sin\beta} - 1\right) = \left(\frac{0.16690240}{0.16690186} - 1\right) = 3.24\ 10^{-6}$$

The average correction to an observed direction is:

$$T = \frac{\frac{1}{2} S\, \rho''}{\Sigma\, \lvert\cot\alpha\rvert + \Sigma\, \lvert\cot\beta\rvert}$$

$$T = \frac{\frac{1}{2} \cdot 3.24\ 10^{-6} \cdot 2.06265\ 10^{5}}{9.366} = 0.''036.$$

With T = 0.″036, as computed from the observed data, check the value given by the Federal Geodetic Control Committee (1974: table 2) for the order of the survey involved. For first order, table 2 shows $T < 0.''3$.

Trilateration

Trilateration is a surveying method in which the lengths of the sides are measured. The availability of electronic distance measuring (EDM) equipment has made this procedure practical and economically feasible. In many instances trilateration provides accuracies that are superior to either conventional triangulation or traverse. Although pure trilateration unsupported by angle measures is seldom used today, the method is explained to complete the discussion of surveying techniques.

The following specifications for trilateration resulted from extensive field tests by the National Geodetic Survey. Most of these tests were carried out with electro-optical instruments on projects involving 1- to 70-km lengths.

In the past, the two principal objections to trilateration were the beliefs that complex figures were required to obtain the equivalent redundancies of triangulation and that additional astronomic azimuths were needed to control the orientation of these networks. However, more recent tests have shown that the single geometric redundancy of the conventional quadrilateral is sufficient for arc-type surveys. Modern EDM instruments eliminate the need to increase the number of azimuths over that specified for triangulation.

To maintain the strength of figure, the quadrilaterals should approximate a square and should seldom contain angles less than 30°. The angle should never be less than 25° for first-order and second-order, class I surveys unless the accuracy of the lengths can be shown to be better by at least 50 percent of the specified standard. In no case, however, should any angles be less than 20° for these classes of accuracy regardless of the amount of improvement beyond the length standard of accuracy.

Central point quadrilaterals with one measured diagonal provide two geometric redundancies for each figure. These are also satisfactory, but for economic reasons should be avoided except in circumstances where both diagonals of the standard quadrilateral cannot be measured. The angle restrictions also apply to the principal chains; however, angles as small as 5° are acceptable for one triangle involving the central point station. In general, the smallest angle in such a triangle should seldom be less than 10°.

Four-sided and other multisided central point figures without diagonals provide a single redundancy, which is satisfactory for all classes of accuracy. In general, the smallest angle restriction specified for the conventional quadrilateral should be applied. The smallest angle should be limited to 35° when figures with five or more sides are involved in first-order or second-order, class I surveys.

For second-order, class II and all third-order surveys limited investigations indicate that the recommended configurations can be significantly modified, if the actual accuracies of the measured lengths are better than the specified standard by a factor of at least 2. In these instances, elongated figures with angles as small as 10°

(or less in some cases) can probably be employed for both classes of third-order work. However, it would be prudent to observe some of the larger angles in each figure.

A chain of single triangles has no check except when ties are made to control points or when azimuths are observed along the route. This type of figure must be used with extreme caution and rarely when survey accuracies above third-order, class I are desired. It is also prudent to observe at least one angle in each triangle.

Geometric restrictions, which limit the selection of locations where stations may be established, hamper the utility of trilateration networks when the purpose is to place control at specially needed sites. These restrictions also complicate the problem of selecting optimum high-ground locations to obtain the necessary instrument heights. The normal requirement for establishing reference and azimuth marks and for determining the positions of intersected objects means that precise angle measurements are required at all stations. It is expected, therefore, that few network surveys will be performed as pure trilateration projects. In recent years, however, measuring the lengths of many additional lines of the conventional triangulation networks has greatly strengthened the network. In some cases this allowed the use of the trilateration technique to strengthen the angular measurements.

Another approach to observing trilateration networks to ultrahigh accuracy is the ratio or relative lateration method. All distances in a figure are measured from both ends using an observing sequence similar to the one employed in securing horizontal angles by the direction method. Because this method is recommended only for surveys that require a superior internal accuracy, no further details or specifications are provided here.

Traverse

Traverse is a method of surveying in which a sequence of directions and distances are observed between adjacent points. Details pertaining to length measurements are discussed in the sections on Electronic Distance Measurements and Number of Length Measurements (p. 14). Traverse design and limitations are discussed under Spacing of Control Points (p. 19).

The following procedure is recommended only when single angles are involved. Although this observational method is not required, good surveying practice dictates the measurement of the angles be made by using one-half of the required number of positions on the odd-numbered circle setting and then observing the exterior angle using the second half of the required positions on the even-numbered circle settings. The sum of the two angles should be in the range of $360° \pm 3''$ for first-order traverses, $360° \pm 4''$ for second-order, class I surveys, and $360° \pm 5''$ for other classes of accuracy. A mean angle referred to a common initial should be compiled for use in the computations. For additional information on procedures and precautions used in high-precision surveys, refer to U.S. Coast and Geodetic Survey *Special Publication* 247, Manual of geodetic triangulation (Gossett 1959). Third-order traverse surveys are described in U.S. Coast and Geodetic Survey *Special Publication* 235, The State plane coordinate systems (a manual for surveyors) (Mitchell and Simmons 1977).

Electronic Distance Measurements

Distances measured with electronic distance measuring (EDM) equipment are subject to errors arising from the instrumental components, calibration of the equipment, inaccuracies in the meteorological data, elevation discrepancies, and the centering of the instruments or reflectors. These factors, as well as the operating procedures, must be considered when deriving specifications for length measurements for a particular standard.

Instrumental errors are usually described in the manufacturer's specifications as a number of millimeters or centimeters, plus or minus a number of parts per million. Various tests have indicated that these statements of accuracy are reasonably valid. However, it must be emphasized that these are average values of results obtained under average conditions, often at a single location, and may not be completely representative of actual field measurements. Nevertheless, these specifications dictate the minimum line length that should be measured with a particular instrument. When measuring long distances, errors are introduced by the inaccuracies of meteorological observations. For short-range measurements, centering the instruments or reflectors and determining elevations for the reduction of slope distances to the horizontal are especially critical.

The minimum distance that should be measured with an EDM instrument depends on four major factors: (1) accuracy required, (2) standard error of a single observation as stated by the manufacturer, (3) number of complete measurements, and (4) observation procedures.

Assuming that precise observing procedures are followed and centering errors are negligible (which may or may not be the case, because extreme care is required to achieve centering accuracies of 1 mm or less), the following formula provides a guide for measuring the minimum required distance:

$$\text{Minimum distance} \cong \sigma/x$$

where σ is the standard error expressed in millimeters and x is a factor that varies with the accuracy class. The values of x are: first-order surveys, 0.005; second-order, class I surveys, 0.010; second-order, class II surveys, 0.025; third-order, class I surveys, 0.050; and third-order, class II surveys, 0.100.

When complete multiple measurements are observed, the acceptable minimum distance can be determined by dividing the value derived in the formula by $\sqrt{N}$, where N is the number of complete measurements. For example, for second-order, class II accuracy, with $\sigma = 5$ mm, the minimum distance is $5/0.025 \cong 200$ m. If two complete measurements are made ($N = 2$), the minimum distance shall not be less than $200\sqrt{2} \cong 140$ m. Some manufacturers state a minimum distance that can be measured. At no time should the stated minimum distance be violated.

Three types of EDM are now in use. These include electro-optical devices that employ visible light, short-range equipment that uses infrared as the carrier wave, and units that rely on microwaves. Microwave equipment and electro-optical instruments, especially those equipped with a laser light source, can measure distances of 100 km or more. Instruments with infrared sources have a limited range, generally 10 km or less.

Some infrared instruments have direct readouts, others indirect. The indirect readout instruments require a series of operations to obtain the slope distance. Their stated accuracies range from 2 to 10 mm, and include 1 ppm times the length (or more) in their statement of accuracy. For crustal movement studies and the establishment of calibration base lines, instruments with a stated accuracy (not to exceed ± 5 mm) should be employed. Measurements for such purposes should be made in a variety of combinations during at least a 2-day period.

A complete measurement using direct-readout equipment consists of ten readings for first- and second-order, class I surveys; five readings for second-order, class II work and third-order, class I and class II trilateration and base lines; and three readings for third-order, class I and II traverses. The spread between the readings should not exceed 5 mm for first- and second-order, class I observations and 10 mm for other classes of surveys. When highest accuracy is specified, two or more sets of observations from both ends of the lines and at different time periods are highly recommended. Differences between complete observations should rarely exceed 10 mm. For requirements of highest precision, this tolerance should not be greater than 5 mm. When only one measurement is specified, it is good practice to observe the distances from both ends of the lines, with the difference between the observations not allowed to exceed 20 mm.

To reduce the possibility of ambiguous results for concentric observations that constitute a complete measurement, it is recommended one observation be made with the instruments or reflectors offset between 0.2 and 0.5 m. If an offset bar is available, this observation can be one of the specified number for the particular class of accuracy. If an offset bar is not available, the instruments or reflectors should be offset between 0.2 and 0.5 m. A rough reading should be taken to ensure that the distance in meters is correct. This precautionary practice is designed to discover ambiguities of 1 m and probably would not detect larger ones. Although such occurrences are uncommon, ambiguous readings of 1 m may occur, particularly if the decimal readings are near zero or in the high nines. Larger ambiguities are rare, but ambiguities of 10 m have been found.

These specifications and permissible tolerances apply generally to measurements obtained with instruments of limited range (3 to 4 km or less). Tolerances for observations made with longer range equipment appear in the section on Number of Length Measurements (p. 14). Some instruments are equipped with a built-in computer to correct the measurements for atmospheric conditions. The corrections should be checked periodically to detect any malfunction in the computer. Experience shows that more accurate and reliable results are obtained by measuring the proper atmospheric conditions and applying the usual formula reductions.

One type of microwave distance measuring instrument can be used under most conditions for short distances (up to 2000 m) with an accuracy comparable to most infrared devices, if extreme care is taken. The instrument also measures long distances by simply changing antenna horns. Although it measures distances in one direction only, an operator is required at each end of the line. When used in the short-range mode, it is recommended that measurements be made from both ends of the line with a spread that does not exceed 20 mm after correcting the measurements for meteorological conditions. When such an instrument is used for short-range measurements, care must be taken within the survey area to avoid various forms of interference that may adversely affect microwaves (e.g., radio transmitters and high voltage power lines).

One high-accuracy, short-range instrument uses a xenon gas-lamp emission source. The instrument has a range of 3000 m; its stated accuracy is about 1 ppm of the distance measured. For projects requiring optimum results, such as special crustal movement studies, dam deformation studies, and high-precision base line measurements, the premium cost for this instrument is justified.

The accuracy of electro-optical instruments is superior to microwave equipment because, unlike microwave instruments, humidity has a negligible effect on electro-optically measured lines. Under some conditions humidity affects microwave measurements by 70 ppm or more. Furthermore, an error of 1°C between wet- and dry-bulb temperatures obtained under normal conditions could produce errors of 10 to 15 ppm in distances measured by microwaves. For distances less than 2 km, the infrared devices usually give superior accuracy and are only slightly affected by humidity.

Excluding the effect of humidity, errors of 1°C in air temperature and 3 mm in barometric pressure (about 30 m on an altimeter) produce inaccuracies of 1 ppm for most electronic distance measuring instruments now in use. The FGCC highly recommends that meteorological data be obtained at both ends of the line for distances measured in connection with all permanently monumented horizontal control surveys. Experience indicates that when the measurements are made at tripod height, temperatures taken at about 10 m above ground level generally are more representative of conditions along the ray path. However, where the ray path barely clears the ground, temperatures obtained at instrument height are usually best. Distance measurements over such terrain, particularly in excess of 5 km, should be avoided; it is questionable whether the temperatures are representative of the actual conditions even if they were obtained at instrument height or 10 m above the ground. Meteorological equipment should always be shaded from the direct rays of the Sun. When highest accuracy is required, instruments should also be protected from the elements (for example, by using observing tents).

For lines exceeding 20 km, where the highest accuracy is required, it is recommended that simultaneous observations be made of the reciprocal vertical angles prior to and after the length observations. When possible, vertical angle measurements should also be made while the observation measurements are in progress. These observations are used to correct the refractive index observed at each end of the line which, when meaned, produce a more representative value of the atmospheric conditions along the ray path. Adverse atmospheric conditions, such as temperature inversion, may cause this correction to be significant for elevation differences as small as 100 m. Therefore, in general this correction should be applied whenever distances are measured for first- and second-order, class I surveys where elevation differences exceed 500 m. To determine this correction, which is actually a combination of two corrections (the second-velocity correction and the index-rate correction), requires that the elevation differences be known, as a general rule, to a few tenths of a meter.

Aircraft equipped with thermisters can also be used to secure very accurate values for the average temperature along a line. This practice is often used to obtain measurements involved in seismic studies. When elevation differences are known to a few tenths of a meter, observing the vertical angles under the terrain situation just described will provide essentially the same information.

An unpublished manuscript, "Corrections for refractive index as applied to electro-optical distance measurements," by B. K. Meade (1969), discusses refractive index. The text explains how aircraft monitor atmospheric data along the measured line.

Multiwavelength instruments now being field-tested indicate distances accurate to 10^{-7} times the distance, or to a few parts in 10^{-8} times the distance, can be determined. This high accuracy is possible because the refractive index is determined by using the differential between the measurements obtained with two or more carriers of distinctly different wavelengths.

Number of Length Measurements

Table 4 lists the number of length measurements commensurate with the various orders and classes of surveys. The narrative portion of the table describes the recommended instrumentation and other specifications for each category (A through G).

Infrared distance measuring instruments may be specified on special projects, such as crustal movement studies, when first-order accuracy is required. In these instances a complete measurement consists of the mean of three observations: one observation made with the reflector centered over the mark and two observations offset ± 0.2 to ± 0.4 m on line. The spread between the three observations shall seldom exceed 5 mm and must never exceed 10 mm.

Electro-optical and infrared measuring devices are recommended for use in subsidiary surveys made to second-order, class I accuracy in areas with high-density population. Two complete observations are suggested, one with the reflector centered over the mark and the other with tbe reflector offset ± 0.4 m. Because lines in such networks are generally short, the spread between the two observations, after reduction to the mark, should not exceed 20 mm. Electro-optical instruments are not recommended where distances to be measured are less than 0.3 km.

The measurement of secondary traverse distances to second-order, class II specifications in metropolitan areas may be determined by using electro-optical or infrared

TABLE 4.—*Commensurate length measurements for designated surveys*
(Letters A to G represent the various classes of accuracy described in the narrative portion of the table.)

Type of survey	First-order specifications	Second-order specifications		Third-order specifications	
		Class I	Class II	Class I	Class II
Base lines	A,B	B	C	D	E
Trilateration	B	C	D	E	F
Traverse	D	E	F	G	G

A. First-order National Horizontal Control Network base lines (1:1,000,000)

Base line spacing: 6 to 12 figures (12 to 25 triangles), depending on geometric strength and configuration of segment of network. These are generally located at junctions of intersection of arcs.

Instrument: *Electro-optical.*

Complete measurement: 4 observations—2 concentric, 2 offset ±0.4 m. (Occasionally a shorter offset bar can be employed.) Spread between observations: Not to exceed 40 mm.

Number of complete measurements: 2 observed on at least 2 days or nights by employing different instruments.

Tolerances: Difference between mean of 2 complete measurements not to exceed 25 mm for lines less than 8 km, and 17 mm plus 1 ppm of the distance for lengths in excess of 8 km.

B. Regular first-order base lines and first-order trilateration (1:1,000,000)
Second-order, class I base lines (1:900,000)

Base line spacing: As required to maintain the scale integrity of the framework, particularly in metropolitan areas usually adhering to R_1 criteria.

Instruments: *Electro-optical* recommended. *Microwave* only when electro-optical not available. *Infrared* for special projects.

Complete measurement: *Electro-optical*, same as specified for "A".

Microwave—A full set of fine and coarse readings, as recommended by manufacturer. For instruments that measure from both ends, the observations at one end shall be completed before observations are begun at the other end. Observations shall be made in the opposite direction when remote unit serves as slave only.

Infrared—Indirect readout: 3 observations—1 concentric and 2 offset, ±0.2 m to ±0.4 m. Direct readout: 10 readings for each measurement—3 observations, 1 concentric and 2 offset, ±0.2 m to ±0.4 m.

Spread between observations: *Electro-optical*, same as specified for "A".

Microwave—Difference between observations made at both ends of line shall not exceed 0.1 m after applying meteorological corrections.

Infrared—Spread between measurements not to exceed 5 mm.

Number of complete measurements: *Electro-optical*—1. *Microwave*—3 on different days or nights, or separated by at least 4 hours between each measurement. *Infrared*—1.

Tolerances: *Microwave*—Spread between 3 complete measurements not to exceed 0.1 m after correcting for meteorological conditions.

C. Second-order, class I trilateration (1:750,000)
Second-order, class II base lines (1:800,000)

Base line spacing: As required to meet R_1 specification.

Instruments: *Electro-optical*, as recommended for second-order, class I trilateration. *Microwave*, only for these survey classes when electro-optical not available. *Infrared* for special projects.

Complete measurement: *Electro-optical*—3 observations: 1 concentric and 2 offset, ±0.4 m. (Occasionally a shorter offset bar may be employed.)
Microwave—Same as specified for "B".
Infrared—Same as specified on "B" except 2 observations: 1 concentric and 1 offset ±0.2 m to ±0.4 m.

Spread between observations: *Electro-optical*—Not to exceed 50 mm.
Microwave—Same as specified for "B".
Infrared—Not to exceed 5 mm.

Number of complete measurements: *Electro-optical*—1.
Microwave—Same as specified for "B".
Infrared—1.

Tolerances: *Microwave*—Same as specified for "B".

TABLE 4.—*Commensurate length measurements for designated surveys*—Continued

D. First-order traverse (1:600,000)
Second-order, class II trilateration (1:450,000)
Third-order, class I base lines (1:500,000)

Base line spacing: As required to meet R_1 specifications.

Instruments: *Electro-optical*—Recommended for first-order traverse.
Microwave—May be used for first-order traverse when electro-optical not available.
Electro-optical, microwave, and *infrared*—Recommended for the other two orders of accuracy.

Complete measurement: *Electro-optical*—For first-order traverse, same as specified for "C". For other classes of accuracy, 2 observations: 1 concentric and 1 offset ±0.4 m. (Occasionally a shorter offset bar may be used.)
Microwave—Same as specified for "B".
Infrared—Same as specified for "C".

Spread between observations: *Electro-optical*—For first-order traverse, same as specified for "C". For the other two orders of accuracy classes 60 mm.
Microwave—Same as specified for "B".
Infrared—Seldom to exceed 5 mm and never to exceed 10 mm.

Number of complete measurements: *Electro-optical*—1.
Microwave—2.
Infrared—1.
Tolerances: *Microwave*—Difference between complete measurements not to exceed 0.1 m after correcting for atmospheric conditions.

E. Second-order, class I traverse (1:300,000).
Third-order, class I trilateration (1:250,000)
Third-order, class II base lines (1:250,000)

Base Line spacing: As required to meet R_1 specifications.

Instruments: *Electro-optical, microwave,* and *infrared.*

Complete measurement: *Electro-optical*—Same as specified for the two lower orders of accuracy in "D".
Microwave—Same as specified for "B".
Infrared—Same as specified for "C".

Spread between observations: Electro-optical—Same as specified for the two lower orders of accuracy in "D".
Microwave—For second-order class I traverse, same as specified for "B". For the two lower orders of accuracy, the difference between observations made at both ends of a line shall not exceed 0.15 m after correcting for atmospheric conditions.
Infrared—Same as specified for "D".

Number of complete measurements: Electro-optical—1.
Microwave—2 for second-order, class I traverse. 1 for the two lower orders of accuracy.
Infrared—1.

Tolerances: *Microwave*—For second-order, class I traverse, differences between 2 complete measurements not to exceed 0.15 m after correcting for atmospheric conditions.

F. Second-order, class II traverse (1:120,000)
Third-order, class II trilateration (1:150,000)

Instruments: *Electro-optical, microwave,* and *infrared.*

Complete measurement: *Electro-optical*—1 concentric.
Microwave—Same as specified for "B".
Infrared—1 concentric.

Spread between observations: *Microwave*—Same as specified for the two lower orders of accuracy in "E".

Number of complete measurements: 1 for all instruments. A check measurement from on an offset position (±0.2m) should be made for infrared instruments when decimal readings are near zero or in the high nines.

G. Third-order, class I traverse (1:60,000)
Third-order, class II traverse (1:30,000)

Instruments: *Electro-optical, microwave,* and *infrared.*

Complete measurement: Same as specified for "F" except when direct readout instruments are employed. Three readings are sufficient for this equipment.
Electro-optical—Distances seldom less than 0.3 km.
Microwave—Spread between measurements seldom to exceed 0.2 m after correcting for meteorological corrections.

Number of complete measurements: Same as specified for "F".

NOTE: The specifications on complete measurements for electro-optical instruments are for those instruments requiring calibration tables. When employing instruments that do not use calibration tables, and more than two observations are specified as a complete measurement, the number of required observations can be reduced by 1.

equipment. However, electro-optical instruments are not recommended for lines of 200 m or less. A single complete measurement with either instrument should be sufficient. The verification check described for use with infrared devices should be made when conditions warrant. For direct readout instruments, it is recommended that lines be measured from both ends to reduce the possibility of blunders. Microwave equipment is not recommended in these situations except for distances in excess of 2 km.

For first- and second-order, class I surveys, distances less than 100 m should be taped using the precise methods outlined in the section on Taped Base Lines or Traverses. For second-order class II, and third-order, class I and II surveys, consideration should be given to measuring distances less than 50 m by using the taping procedures recommended for the particular order of work.

All electronic distance measuring devices should be serviced regularly and checked frequently over lines of known distances. Instruments should be calibrated at least annually, preferably every 6 months. Frequency checks are recommended every 3 to 4 months. Equipment must be handled with care and protected from the elements at all times. When the highest accuracy (better than 1:500,000) is required, frequencies should be monitored throughout the observations. For accuracy requirements between 1:300,000 and 1:500,000, crystal frequencies should be checked daily.

The National Geodetic Survey has established specific calibration base lines to test EDM instruments and maintains data listings for these calibration base lines. For information, contact: National Geodetic Information Center, NOS/NOAA, Rockville, MD 20852. *NOAA Technical Memorandum* NOS NGS-8, Establishment of calibration base lines (Dracup et al. 1977), and *NOAA Technical Memorandum* NOS NGS-10, Use of calibration base lines (Fronczek 1977) provide further instructions.

Taped Base Lines or Traverses

The general availability of EDM equipment has drastically reduced the use of taping procedures for measuring base lines or traverse lengths. New advances in EDM technology, particularly the capability to measure shorter distances with higher accuracy, will further reduce the need for taping. Probably some need will always remain to use taping procedures for the following: calibration base lines, short traverse connections, engineering or scientific projects, and for minimum cost, low-order property surveys. The previous section on EDM measurements discusses those situations where taping should be performed to meet accuracy specifications.

Coast and Geodetic Survey *Special Publication* 247, Manual of geodetic triangulation (Gossett 1959) contains detailed procedures on taped measurements of high-order accuracy for base lines and traverses. Procedures for performing lower-order surveys are determined on the basis of accuracy required. As a minimum, stakes (or taping bucks) and standardized tapes, or tapes previously compared with a standard length, are recommended. For second-order, class I or higher standards, the tapes should be made of low-coefficient nickel steel (invar). Accurate differences of elevation between tape supports are required to reduce the measurements to the horizontal plane.

The reduction of taped lengths must include corrections for: (1) the difference between field observed temperatures and the temperature used in the standardization or comparison, (2) the "true" length of the tape, (3) the slope of the line, (4) the support condition, and (5) in most instances, reduction to the ellipsoid.

The reduction of lengths to sea level should not be confused with the reduction to the ellipsoid. Historically, the correction for the reduction from sea level to the ellipsoid has been ignored because of insufficient geoid height information. However, for very precise work or for geodetic datums, such as the forthcoming North American Datum 1983, where the two surfaces (sea level and the ellipsoid) are not nearly coincident, an additional reduction from sea level to the ellipsoid should be applied.

Astronomic Azimuths

The accumulation of angular errors in horizontal control surveys and a tendency for a portion of these errors to be somewhat systematic (thus providing the opportunity for the development of a twist in the direction of a survey), requires the inclusion of astronomic azimuth observations in the surveys at specified intervals. For precise surveys the astronomic observations must include a determination of longitude. Theoretically, because the astronomic latitude is required in order to compute the azimuth and to define completely the deflection of the vertical, the astronomic latitude is also observed.

The astronomic longitude is used in determining the correction for the prime vertical component of the deflection. This value is then used to compute the Laplace correction, which converts the astronomic azimuth to a geodetic azimuth. Values for the deflection of the vertical can be interpolated from previously observed determinations for many areas of the United States. Except for some anomalous locations, these interpolations are sufficiently accurate for the correction of azimuth observations used to control second-order, class II and some higher-order traverses. By using sophisticated computer programs, both components of the deflection of the vertical can be predicted to about 1″ if sufficient gravity coverage exists in the area.

Department of the Army Technical Manual TM 5-442, Precise astronomic surveys (U.S. Army 1970), and U.S. Coast and Geodetic Survey *Special Publication* 237, Manual of geodetic astronomy (Hoskinson and Duerksen 1952), contain discussions of the Laplace correction and methods for determining azimuths.

More accurate azimuths may be obtained by making simultaneous observations at both ends of the lines. This practice usually cancels the effect of a large portion of the lateral refraction. For surveys requiring the highest accuracy, the observations should be obtained by using at least two observers and at least two instruments. Each observer should observe two sets each night for three nights. Observations should be made during the entire period of darkness. Rejections and criteria for reobservations must be prescribed so as to ascertain an accuracy of better than 0″.5.

Reconnaissance, Site Selection, and Property Ownership

Reconnaissance for horizontal control surveys should be performed well in advance of the arrival of the observing party. Instructions for the reconnaissance should define the area to be covered, the survey accuracy specified for the project, the desirable spacing of control points, connections to existing surveys, and any special requirements for station locations including, on occasion, those sites where particular base line and astronomic observations are to be made. Information should include any contracts, negotiations, or preliminary planning that involves another organization.

As a general rule, reconnaissance personnel are responsible for satisfying all requirements for strength of figure, line of sight, and other technical specifications. Before a survey can begin, permission must be secured from property owners or managers, and special agreements negotiated if necessary.

An office review of the proposed survey should then be made to determine if all requirements have been satisfied. Select sites where the base lines will be measured and the astronomic observations obtained. When the survey involves "long packs" or helicopter support to reach the station sites, simulated studies should be undertaken to confirm that the desired standard would be the most economical. This approach is also recommended where the configurations may be geometrically weak or of minimum strength. However, in most cases when the primary doubts are related to the sufficiency of scale or orientation of a survey, it is often simpler to specify that additional lines be measured or astronomic azimuths observed rather than perform the computational studies. U.S. Coast and Geodetic Survey *Special Publication* 225, Manual of reconnaissance for triangulation (Mussetter 1959) provides details for executing a reconnaissance survey.

Connections to Existing Surveys

When connecting first- or second-order, class I surveys to established control, new measurements of previously observed angles must be made to check the recovery of these stations. If the new observations fail to check the previously observed values by more than 2″, the observations should be repeated and all field conditions that might cause such changes investigated. The National Geodetic Survey should be notified. It is possible that justification exists for accepting check angles that exceed this maximum. The length of at least one previously observed line should be measured near the initial and terminal stations to furnish additional scale for the existing triangulation or to act as an additional check on the recovery of the station.

An integral part of the recovery of a station involves the reobservation and remeasurement of the directions and distances to the reference marks. The directions to the azimuth mark and to previously observed landmarks, such as water tanks and church spires, should be included in the reobservation program. When significant differences are found, the newly obtained values should be verified in the field and NGS notified. The value for a significant difference may vary under certain circumstances. In general, whenever the directions or distances to the reference marks cannot be checked within 3′ and 1 cm, respectively, and the directions to the azimuth marks and objects fail to agree within 10″, specific mention of these discrepancies should be noted in the recovery notes.

For second-order, class II or lower-order surveys, it would be desirable to follow the specifications cited here. If cost or time factors make this recommendation prohibitive, a minimum requirement would be the verification of the distances and directions to the reference marks. If other stations or intersection points are visible, observations should be made to these points and compared with previously observed or computed values. It is much better to orient a survey through azimuths to other distant stations, which were previously observed as part of the existing survey, than to use a nearby azimuth mark.

New surveys should normally begin and terminate at control points with an accuracy equal to or higher than that required for the new network. This does not preclude connecting higher-order surveys to lower-order surveys in special cases. For example, in a scientific or engineering project, where the absolute geographic positions are relatively unimportant, a high internal accuracy may be required. Whenever practical and economical, surveyors are requested to contribute data from all their control surveying activities to NGS for inclusion in the National Networks of Geodetic Control. In this instance, prescribed connections to the lower-order surveys are acceptable. To obtain internally consistent data the published position of one previously established point is held fixed, and a minimum constrained adjustment of the new observations is performed. On occasion, the new observations may be used to strengthen the lower-order surveys in the vicinity through a readjustment of the old observations. New observations can also be retained for future use.

Established stations along a survey route should not be bypassed simply because they may be difficult to reach or require additional effort to connect. The "20 percent rule," described in the next section (p. 20), must be rigorously followed. Many problems with the present national network and with numerous local systems resulted from the omission of connections to established control at the time the nets were observed and adjusted, even though the connections were included in subsequent surveys.

Spacing of Control Points

Horizontal control points may be spaced at any required interval. Such spacing is restricted only by the limitations of available instrumentation and technical procedures.

In triangulation and trilateration, the lengths of lines are usually governed by the requirements of the project instructions, topographic conditions, and strength-of-figure considerations. In general, national network first-order stations are seldom spaced less than 15 km apart; second-order, class I control points, 10 km; and second-order, class II control points, 5 km. Spacing of survey projects in areas of intensive economic development may be much closer, as stated in *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1974: table 2). Project instructions issued by Federal agencies usually require that points be located at airports, colleges, towns, along waterways, and principal highways. Whenever practical, connections should be made to horizontal and vertical monuments established by other surveying and mapping agencies. These can be main scheme stations if the strength of figure is maintained and, in the case of existing marks, the monumentation is satisfactory. Otherwise, new control should be established.

When selecting locations for horizontal control points, consideration should always be given to present or future the control extension requirements. Therefore, as a part of the observing schedule, prominent landmarks or natural objects should also be positioned by third-order intersection methods.

As a general rule, the spacing of traverse stations is restricted only by the minimum length-of-line specification. However, traverses have little or no inherent geometric strength, such as that found in triangulation, and, to a lesser extent, in trilateration. Design factors that weaken traverse networks must be eliminated or minimized. Specifically, traverses—whether a single-line type or interconnected networks—should be designed so the routes between control stations are more or less straight lines with the distances between new stations (courses) of about equal length. Loop systems should be essentially square or rectangular. For first- and second-order, class I traverses, any deviation greater than 20° from a straight line should be avoided. It is important that the courses be of nearly equal length.

When topography or other conditions necessitate the introduction of abrupt changes in the routes, or require shortened or expanded course lengths, additional observations should be considered. These included observation of astronomic azimuths in the vicinity of the abrupt changes, reobservations of the angles at these sites, or remeasurements of the distances.

The specifications for traverse networks of higher accuracy cited here ensure that the components involved in position closures are directly related to the observational element involved in the particular component. For example, the closure in latitude in an east-west traverse should be directly related to the angulation or orientation, whereas the closure in longitude should be length-related. Whenever abrupt changes or short courses are interspersed with longer lines, it is extremely difficult to evaluate a traverse network. For this reason strict requirements must be specified.

The restrictions also apply to second-order, class II traverses, where the stations are spaced to national network specifications and when the purpose of the survey is to supplement the national network. However, some leeway is permitted for this survey class in areas where topography requires using equipment not readily available or where development is unlikely in the near future.

Metropolitan area subsidiary (breakdown) surveys often involve stations spaced at intervals of 200 to 2000 m. Where second-order, class I and class II standards are desired for such surveys, the design patterns described previously should be followed. Because this pattern will not always be possible, design modifications can often be justified.

The major problems encountered in traverses involving short lines are those related to the accuracy of orientation and the availability of EDM equipment. Problems involving very short distance measurements can be resolved by employing taping procedures. Those involving orientation can be reduced by instrumentation (e.g., the use of "forced centering systems"), the insertion of astronomic azimuths at more frequent intervals, or the reobservation of the angles on at least one other occasion.

When closely spaced stations are included to meet project requirements rather than topographic limitations, it is often possible to maintain overall accuracy by measuring between more widely separated stations. The more closely spaced points can then be located by closed loop traverses that include these stations.

Where somewhat parallel traverses or surveys established by other methods approach each other, they should be connected if practical. The 20-percent rule is applied in this instance. This rule, formulated by Lansing Simmons of the former U.S. Coast and Geodetic Survey, states that whenever the distance between two unconnected points is 20 percent (or less) of the sum of the distances between directly connected stations involved in the shortest route between the two points, a connection between the points should be made if possible and practical (Dracup 1979: 29-30). The rule is applicable to all types of horizontal control networks and must be adhered to for first- and second-order, class I connections.

Ideally, multiple traverses in an area should be interconnected to form a somewhat rectangular pattern and all should be included in a single adjustment.

Strength of Figure

The strength-of-figure determination is an expression of the comparative precision of computed lengths in a triangulation net as determined by the size of the angles, the number of conditions to be satisfied, the distribution of base lines, and the lengths determined in previous adjustments. Strength of figure in triangulation is not based on an absolute scale but, rather, is an expression of relative strength.

Although the strength of figure concept remains an important consideration in performing triangulation reconnaissance, its use in determining the need for base lines has been relegated to a less prominent role. With the increased use of electronic distance measuring equipment, and hence the ability to measure distances almost anywhere, its essentially economic reliance has been almost eliminated. As modern survey networks become a mixture of triangulation, trilateration, and traverse, other evaluation measures, which are much more complex than the simple strength-of-figure test, are required to ascertain the acceptability of the design. Nevertheless, strength-of-figure tolerances are still an integral part of the specifications for triangulation which, for the immediate future, continue to be one of the primary means for establishing primary national network control.

Length, Azimuth, and Position Closures

In theory, these closures should be calculated after the geometric conditions for triangulation and trilateration, and the azimuth closure for traverse have been accounted for in a minimum constrained adjustment. However, if the computations are performed in the field or if computers and adjustment programs are not available, the following practices provide results which, as a rule, are good approximations of those obtained from minimum constrained adjustments.

For triangulation surveys, the triangle closures are distributed equally on each angle of the respective triangles and the sides are calculated. The length closure is obtained by computing through the strongest chain or chains to the lines of known length. Geographic positions are then computed over the same chains to determine the azimuth and position closures. For trilateration, the measured terminal sides furnish the length checks. Azimuth and position closures are secured by computing through the strongest chain or chains of triangles using the computed angles and measured lengths. In traverse, the azimuth closures are calculated by preliminary computations using either an initial grid azimuth or geographic positions. On first consideration, the use of an initial grid azimuth seems the simplest procedure; however, the observed angles should be corrected for the arc-chord (t-T or second term) corrections, which involves additional effort even when using approximate formulas. The distribution of the azimuth closures in traverses, as gained from long experience, is best handled by applying the first closure in equal amounts to the traverse angles, taking the route that contains the fewest number of angles between fixed azimuths. The second and succeeding closures are distributed in the same fashion using the corrected azimuths, where applicable, from previous computations. When a survey includes a combination of methods, a careful review and judicious utilization of the procedures described will generally provide an acceptable basis for judging whether the closures meet the criteria.

Elevations of Horizontal Control Points

Whenever distance measurements are made in the course of performing control surveys to provide scale or, as is the case in trilateration and traverse, a fundamental element in the positioning of points by these methods, it is necessary to reduce the measurements to the horizontal and to the mean sea-level surface (geoid). For highest accuracy, it is often advisable to reduce distance measurement to the reference ellipsoid. (See Taped Base Lines or Traverses, p. 17). For first- and second-order, class I surveys, the use ot elevations is the preferred method. Although accurate elevations are desirable, the accuracy of the *differences* of elevation (as opposed to absolute elevations) is of greater concern, especially where these differences are responsible for slope corrections in excess of 0.1 m. The reductions are computed using the elevations of the points at which the measurements were made, plus the accurately measured heights above the stations of the instruments or reflectors.

It is preferable to determine elevations by spirit or compensator leveling methods, either directly (that is, the stations also serve as bench marks in a leveling line) or indirectly by connections to the bench marks. Because most control stations are located in outlying areas, direct ties are not usually possible. Most of the elevations for these points are determined by trigonometric leveling using zenith distance observations—a form of vertical angles—in the computations.

Although the use of elevations to reduce measurements is highly recommended for second-order, class II surveys and surveys of lower accuracy, particularly those involving trilateration, vertical angles may be used for the reduction to the horizontal. Caution must be exercised when the cosine of the vertical angle is used to reduce slope measurements to the horizontal. If the observations are made only at one end of the line, the correction for curvature and refraction should be applied as a general practice, even if it does not significantly affect the cosine. This correction is not necessary when observations are made at both ends of the line and a mean vertical angle is used. In addition, any appreciable difference (and this can be as little as 0.3 m) in the heights at which the vertical angles and the length measurements were secured must be taken into account as another correction to the observed vertical angle. The best procedure is to compute the elevations of the marks at the two ends of the line by the established zenith distance (vertical angle) methods (reciprocal or otherwise), taking into account instrument and target height. The resulting elevations are then used to convert the slope lengths to the horizontal and/or to mean sea level distances by standard procedures.

Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys (Federal Geodetic Control Committee 1974: table 2) provides a limitation on the number of figures over which trigonometric leveling can be carried before a check on the elevations is required. This check may be made against previously determined trigonometric elevations derived from observations obtained using similar specifications, as required for the particular survey or elevations from spirit or compensator leveling. For those projects where the elevations are required only to reduce isolated base lines, the elevation of one of the base line stations may be scaled from a high quality map or, in some cases, determined by a well-adjusted barometer or altimeter. This procedure is permissible when connections to points of known elevation would involve considerable effort. The difference of elevation between the stations at each base line site could then be determined by spirit or compensator leveling or reciprocal vertical angles. When employing these methods, remember that a change of 6.4 m in elevation will affect the sea-level or ellipsoidal reduction by 1 ppm, a change of 12.8 m will be affected by 2 ppm, etc.

Elevations are also required to reduce horizontal directions observed at higher elevations and in mountainous regions, and to reduce astronomic observations to the reference surface.

The observational and computational procedures for vertical angle measurements for triangulation and trilateration are discussed in U.S. Coast and Geodetic Survey *Special Publication* 247, Manual of geodetic triangulation (Gossett 1959: 15, 103, 127, 139, 143, 184) and U.S. Coast and Geodetic Survey *Publication* G-56, Elevations from zenith distances (Poling 1947). Although procedures for traverse are essentially the same, special emphasis must be placed on observations over the short or steep lines, which are more common to these surveys. Because redundant observations are lacking, extra care must be exercised.

The number of required vertical angle observations is published in *Classification, Standards of Accuracy, and*

General Specifications of Geodetic Control Surveys (Federal Geodetic Control Committee 1974: table 2, columnar heading: "classification vertical angle observations"). A single observation consists of one direct and one reverse pointing. If the permissible spread between the observations in a set is exceeded, the observations must be repeated until a satisfactory set is obtained. The heights of the instrument and target reflectors above the marks must be accurately measured. Special care must be exercised where the difference in elevation divided by the distance exceeds 0.03. When this ratio approaches 0.1 and the distances are less than 300 m, consideration should be given to leveling between the points. Refraction is a major consideration in obtaining accurate vertical angle observations; observations should be made when atmospheric conditions that cause vertical refraction are the most stable. In most areas the best time period for vertical angle observations is between 10 a.m. and 4 p.m. When the highest accuracy is required, reciprocal vertical angle observations should be made as nearly simultaneously as possible during the 10-a.m. to 4-p.m. period.

Other Sources of Error

The specifications stated herein include requirements for instrumental accuracy, the number of necessary observations, and the required computational closures which must be satisfied to give reasonable assurance the stated standard is met. Some of the other sections mention additional precautions. If these precautions are not taken, lack of action may prevent the attainment of a desired accuracy for a project, even if all major requirements have been met. The following procedures are precautionary suggestions rather than part of the formal specifications.

Collimation and Eccentricity

These are related problems that affect the accuracy of both angle and distance measurements. They may generally be categorized as the *centering* of instruments and targets. The effect of erroneous centering on distance measurements is direct and rather obvious. For angulation, however, the importance (especially on short lines) must not be overlooked. With reasonable care, angles can be measured to a precision less than 1".0 over lines of any length and an accuracy (3 sigma) approaching this value for lines in excess of 400 m. With special care, and in controlled situations, angles accurate (3 sigma) to 0".5 and less are possible. This is meaningless if the instrument or targets are not centered over the marks and sufficient measurements are not obtained to reduce the observations to the true points.

As a general rule, instruments and targets can be positioned within about 1 mm of the true center. An instrument or target may be deliberately or accidentally operated or observed in an eccentric position, which is usually a short distance from the true station. Accurate measurements of the angle and distance involving the eccentric and true points must be obtained in order to reduce the observations to the true point or to maintain the relationship of the points. The following examples give some indication of the magnitude of angular errors caused by small miscenterings. A 3-mm error in centering a target causes a 0".4 error at 1500 m and 4" at 150 m. For traverse points spaced at 150 m, if both the target and instrument are miscentered by 3 mm, the angle error could exceed 16". The following formula gives the maximum error in seconds of arc resulting from miscentering a signal by 1 mm: $E'' = 206$ S, where S is the distance in meters.

Errors caused by a lack of collimation adjustment in theodolites, astronomic transits, and even precise levels are usually eliminated by symmetrical observations in which the collimation error is introduced equally with opposite signs. This is achieved in theodolites and astronomic transits by observing with the telescope in both the direct and reverse positions, and in precise leveling by balancing the sight distances.

Stability and Rigidity of Supports

In precise surveys the towers, stands, and tripods must be stable. The use of driven stakes or some type of quick setting cement for tripod leg supports may be required. Catwalks, supported away from the tripod legs, may also be necessary under some soil conditions to ensure satisfactory results. Regular instrument tripods should rarely be used in first- or second-order, class I surveys.

Phase Error

Phase error that occurs in horizontal angle measurement is attributable to an apparent displacement of a target. Such displacements can be caused by unequal daylight illumination of a target or by lighted targets that are not pointed directly toward the observer. For precise surveys, the use of carefully pointed illuminated targets is recommended for both day and night operations. The effect of phase error is the same as the effect described for centering errors, except it is not often readily apparent. Even when detected, phase error is not easily corrected mathematically.

Refraction

A ray of light bends as it passes obliquely through air strata of different densities. Most refraction of this type is in the vertical plane, but frequently there is a measurable component in the horizontal plane. Horizontal refraction is one of the most uncertain factors encountered in the measurement of horizontal angles. Night observations have occasionally been in error by 5″ or 6″ and daylight observations by two or three times these figures. Corrections for refraction cannot be applied to observations, but a careful reconnaissance, avoiding suspicious topographic features and selecting the best available observing conditions, will usually eliminate or substantially reduce this effect. For example, refraction errors that appear in the form of grazing lines often occur when the instrument's line of sight is parallel to a sloping hillside and the wind is simultaneously blowing away from the slope. Another condition to avoid is the presence of obstructions near the ends of the lines, as they might "bend" the line of sight. U.S. Coast and Geodetic Survey *Special Publication* 247, Manual of geodetic triangulation (Gossett 1950: 135) describes horizontal refraction.

Shelter

Protecting instruments from uneven thermal expansion and from wind vibration is often required for precise surveys. Minimum protection should consist of a large umbrella to block the Sun's rays plus a wind screen if needed. A full tent is recommended for lengthy observation schedules under adverse weather conditions.

Monumentation and Description

All first- and second-order, class I horizontal control stations shall be monumented and described. In addition to the station mark, monumentation also includes an underground mark where possible, two or more reference marks, and an azimuth mark. Another horizontal control point or, occasionally, landmarks visible from the ground may be substituted for an azimuth mark. Although observations to all objects, or to a representative grouping of man-made or natural structures, visible from tripod height at a station should be included in the station description, establishment of a monumented azimuth mark is usually required. Many of these objects are subject to repairs or rebuilding in approximately the same location, displacements caused by aging, and, in the case of radio masts and other guyed structures, movement caused by tightening, replacing, or adding guy wires or sections. For these reasons, such objects should seldom be used in place of a monumented azimuth mark. To orient surveys, observations computed to landmarks or natural objects are preferred over observations made to nearby azimuth marks. However, these landmarks or natural objects must be positively identifiable and located at a substantial distance from the observer. In addition, the observed objects must be well defined and have known (or computable) azimuths.

These specifications are applicable to stations established to second-order, class II standards. But, as noted previously, an occasional point marked in a temporary fashion, particularly for traverses, is permissible in some circumstances. However, such occasions should be rare, as it is not economical or proper to obtain quality results without correct permanent monumentation and descriptions.

Third-order monumented stations should be located in protected areas, if possible, and should be described with sufficient care to enable recovery at a future date. Landmarks and natural objects determined to third-order accuracy by intersection or traverse connection methods should be described with adequate care and detail to give reasonable assurance that the points may be positively identified later.

When emplacing marks, the surveyor must consider problems related to settlement in soft soils, possible movement caused by freezing and thawing, long-term erosion and wave action, and other natural or cultural effects.

A witness post or sign should be placed near each monument if local conditions permit to aid recovery and to protect the marker against accidental destruction.

The station description shall include information on how to reach the general location of the station site from a town or from some prominent feature normally displayed on maps. A detailed description is written of each monument, relating the location to nearby permanent objects. Distances and directions to the reference marks relative to the station mark shall be included.

These observations and measurements shall be determined with sufficient accuracy in order to: aid in the recovery of the underground mark in the event the surface mark is destroyed, replace the station in its original location, establish a new station nearby, or permit a reference mark to act as a substitute station.

U.S. Coast and Geodetic Survey *Special Publication* 247, Manual of geodetic triangulation (Gossett 1950) and *ESSA Technical Memorandum* C&GSTM-4, Specifications for horizontal marks (Baker 1968) describe the methods recommended for constructing monuments, including station marks and designations.

Vertical Control Surveys

The National Vertical Control Network consists of a hierarchy of interrelated nets which span the Nation. Table 5 summarizes the classification of the various components. The most crucial elements of this net are:

(1) A set of large circuits (loops) at an average grid size of 100 to 300 km (shown in fig. 2 as basic net A).

(2) A subdivision of circuits with an average grid size of 50 to 100 km, identified as basic net B.

(3) A secondary densification net with a line spacing of 10 to 50 km. This net provides additional vertical control points, which are determined using slightly lower accuracy standards.

Each level of the hierarchy is adjusted to conform to net elevations having equal or higher accuracy. Elevations computed from a new survey cannot be given a classification higher than that of the elevations which were fixed to define the datum relationship in the adjustment. These primary and secondary nets provide a common reference system for users' needs. The total net is shown in figure 3.

A continuing program of mark maintenance replaces destroyed marks, relevels older lines of the primary net, and extends control into areas having inadequate coverage. This activity maintains the national vertical control net at a level of accuracy and availability consistent with users' needs.

Bench mark elevations of the National Vertical Control Network are currently being published as normal orthometric heights. (See U.S. Coast and Geodetic Survey *Special Publication* 240, Manual of leveling computation and adjustment (Rappleye 1948a: 155).) Elevations are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29) (Federal Register 1976). Prior to September 1973, publications used the designation "Sea Level Datum of 1929." The 1929 datum was established by constraining the combined United States and Canadian first-order leveling nets, as they existed in 1929, to conform to mean sea level (MSL) of various epochs. These epochs were determined using data gathered from 26 long-term tide stations along the Atlantic and Pacific coasts and the Gulf of Mexico (21 in the United States and 5 in Canada). Observed spirit level differences of elevation, referenced to the zero elevation of MSL at the 26 tide stations, were interconnected to form a net of approximately 103,000 km of first-order level lines. These observations, after applying orthometric corrections, corrections for other known systematic errors, and simultaneous adjustment of loop misclosures, provided a set of consistent orthometric elevations for all bench marks in the adjusted system. This adjustment was called the "General Adjustment of 1929." The resulting elevations defined the system which (with a few exceptions) was the basis of all subsequent geodetic leveling in the United States. By 1980, a total of 700,000 km of new first- and second-order leveling had been adjusted to this system.

First-Order, Class I (Primary Vertical Control)

First-order, class I leveling is primarily used for basic net A. (See table 5.) It is also used for metropolitan area control, regional engineering, and crustal movement projects. Line spacing for basic net A is 100 to 300 km, depending on population density and use. The maximum length of line between net junctions is 300 km. Bench mark spacing averages 1.6 km. Maximum separation does not exceed 3 km. Closer spacing is appropriate for more densely populated areas. Bench mark spacing is reduced to less than 1 km along steep slopes in order to avoid setting temporary bench marks. Sections are observed either in both forward and backward directions, or in one direction when using the double-simultaneous mode. Gravity values are required at bench mark settings if elevation differences and elevations are to be computed in geopotential units (gpu). Gravity values can either be observed or interpolated from nearby observed gravity, but the effect on geopotential heights is not allowed to exceed the limits stated in table 6.

First-Order, Class II (Primary Vertical Control)

First-order, class II leveling is most often used in establishing basic net B, and for releveling portions of basic net A. It is also employed for metropolitan area control, regional engineering, and crustal movement studies. Line spacing on basic net B is between 50 to 100 km, with a maximum distance between junctions of 100 km. Tables 5 and 6 summarize mark spacing, instrumentation, section runnings, and gravity requirements.

Second-Order, Class I (Secondary Vertical Control)

Vertical surveys of second-order, class I accuracy are used for the secondary net, for metropolitan area control, and for large engineering and crustal movement projects. Line spacing is 20 to 50 km, with a maximum distance

TABLE 5.—*Synopsis of classifications for vertical control surveys*

Attributes	Orders of surveys and classes of accuracy: First-order class I	First-order class II	Second-order class I	Second-order class II	Third-order —
Principal uses	Basic net A	Basic net B	Secondary net	Area control	Local control
Line spacing	(km)	(km)	(km)	(km)	(km)
National net	100-300	50-100	20-50	10-25	As needed
Metropolitan	2-8	2-8	0.5-1	As needed	As needed
Maximum length of line between junctions	300	100	50	50, double-run leveling 25, single-run leveling	25, double-run leveling 10, single-run leveling
Mark spacing	Average 1.6, maximum 3	Average 1.6, maximum 3	Average 1.6, maximum 3	Maximum 3	Maximum 3
Section runnings	Double-simultaneous[1] or forward and backward	Double-simultaneous[1] or forward and backward	Double-simultaneous[1] or forward and backward	>25, double-run leveling <25, single-run leveling	>10, double-run leveling <10, single-run leveling
Instrumentation	Compensator or tilting level with parallel plate micrometer and invar double-scale rods	Compensator or tilting level with parallel plate micrometer and invar double-scale rods	Compensator or tilting level with parallel plate micrometer or three-wire levels and invar scale rods	Geodetic level and invar scale rods	Geodetic level and rods
Gravity[2]	Observe or interpolate	Observe or interpolate	Observe or interpolate	Observe or interpolate	Observe or interpolate

[1]Used where it can be evaluated by loop closures.

[2]Required only if elevations are computed in geopotential units.

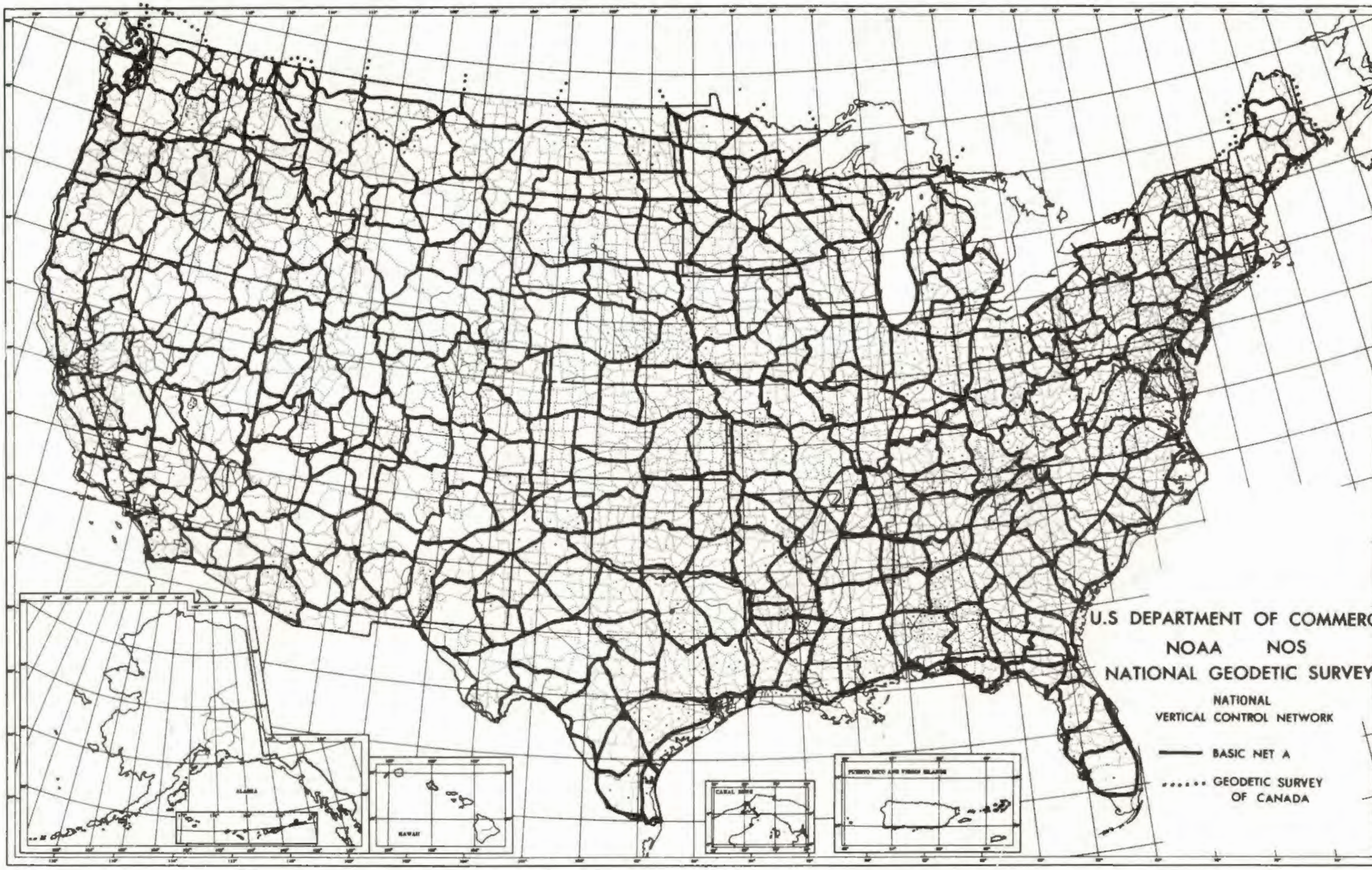

FIGURE 2.—National Geodetic Vertical Control Network.

FIGURE 3.—Status of vertical control in the United States.

TABLE 6.—*Tolerances and factors for vertical control surveys*

Attributes	First-order class I	First-order class II	Second-order class I	Second-order class II	Third-order ——
	Tolerances				
	(mm)	(mm)	(mm)	(mm)	(mm)
Gravity effect	0.30	0.40	0.60	0.80	1.20
	(m)	(m)	(m)	(m)	(m)
Maximum length of sight	50.0	60.0	60.0	70.0	90.0
Maximum difference in length of forward and backward sights:					
per setup	2.00	5.00	5.00	10.00	10.00
per section	4.00	10.00	10.00	10.00	10.00
Low-high scale elevation difference for	(mm)	(mm)	(mm)	(mm)	(mm)
setup	0.25	0.30	0.60	0.70	1.30
Loop misclosure	4.00	5.00	6.00	8.00	12.00
Section, f+b	3.00	4.00	6.00	8.00	12.00

Number of runnings	Section, factors from simple mean				
	(mm)	(mm)	(mm)	(mm)	(mm)
3	2.10	2.81	4.21	5.63	8.44
4	2.33	3.10	4.66	6.23	9.34
5	2.48	3.31	4.96	6.64	9.95
6	2.59	3.46	5.19	6.94	10.4
7	2.68	3.58	5.36	7.18	10.7
8	2.75	3.67	5.51	7.37	11.0

of 50 km between junctions. Instrumentation requirements are the same as for first-order leveling, except three-wire instruments and single-scale invar rods can be used. Tables 5 and 6 show section runnings and gravity requirements.

Second-Order, Class II (Supplemental Vertical Control)

Second-order, class II leveling is used for area control, local engineering, and topographic mapping projects. Line spacing for area control is 10 to 25 km. The maximum length of line between junctions is 50 km for double-run and 25 km for single-run leveling. The maximum line length can be increased to 100 km for double-run leveling in areas where higher order control nets have not been established. Geodetic level instruments and invar rods shall be used for double-run leveling. Sections may be single-run (if less than 25 km) or double-run (if greater than 25 km). Tables 5 and 6 show the gravity requirements.

Third-Order (Local Vertical Control)

Third-order leveling is used for local control, small engineering projects, topographic mapping projects, and other work requiring this accuracy. Line spacing depends on project requirements. The maximum length of line between junctions is 25 km for double-run and 10 km for single-run leveling. The maximum length of a double-run line may be increased to 50 km in areas where higher order control nets have not been established. Tables 5 and 6 show the gravity requirements.

Network Releveling

Elevations of bench marks of the national leveling net are subject to deterioration from crustal motion and surface changes. Construction and land development also contribute to the destruction of bench marks. A continuing program of resurveys is required, with readjustment about every 15 years, to maintain the proper accuracy and accessibility of the net. State and local agencies supplement, to a limited extent, the resurveys required for local projects. In areas with significant crustal movement, resurveys must be performed within a short time span to minimize differential effects within the area and to provide compatible adjusted elevations. State-of-the-art equipment and survey procedures are used when resurveying basic net A. First-order, class I or class II observing procedures are used in the forward and backward or double-simultaneous modes. (See table 5.)

Gravity values are needed at bench marks of the national net to minimize the effect of gravity anomalies. Additional gravity must be observed if interpolated values from existing observed gravity cause an uncertainty in the elevation difference larger than one-tenth of the allowable tolerance between forward and backward leveling. (See table 6.) A 1-mm change in height is approximately equal to 0.001 gpu (1 gpu equals 1 kilogal meter). A 5-mgal uncertainty in average gravity values causes about 0.0005 gpu uncertainty in a 100-meter elevation difference.

Tabulated factors (table 6) for loop, forward and backward (f+b), and section limits from the simple mean must be multiplied by the square root of the loop or section length in kilometers to obtain the proper tolerances. Loop misclosure limits apply to misclosures computed using new or previous leveling of the same order and class. Crustal or surface movement may cause unacceptable loop misclosures when compared against previous leveling. When loops are composed of segments of different orders and classes of leveling, the tolerance for the loop misclosure is the square root of the sum of the squares of the tolerances of the individual segments (links) of the loop. For example, for a loop consisting of 25 km of first-order, class I; 49 km of second-order, class I; and 36 km of third-order leveling, the allowable loop misclosure would be

$$\sqrt{4^2 \times 25 + 6^2 \times 49 + 12^2 \times 36} = 85.7 \text{ mm}.$$

Section f + b tolerances apply to the absolute value of f + b runs for a section. The f + b values are also summed algebraically along each level line, to aid in detection of systematic error. For first-order, class I leveling, use only sets of forward and backward runs when the mean difference is computed, unless the refraction correction is computed for each running.

When more than two runnings are observed, the simple mean of all runnings, and the absolute value of the difference between each running and the simple mean, are computed. The factor taken from table 6, based on the number of runnings and the order and class of the survey, is multiplied by the square root of the section length in kilometers to obtain the tolerance. The rejection process is iterative. For each iteration only the running having the largest deviation from the simple mean is tested. If the amount of deviation exceeds the tolerance, the running is rejected. A new mean is then computed before the running is tested with the next largest deviation. The process continues until all retained runnings are acceptable, or until one forward and one backward running remain. In the latter case, the section f+b tolerance limit is applied. If all forward or all backward runnings are rejected in double-run leveling, new runnings must be observed until at least one forward running and one backward running are retained after applying the rejection process. When computing a tolerance limit for section runnings of less than 0.1 km, multiply the factor in table 6 by 0.316. The section elevation difference is the simple (indiscriminate) mean of the absolute value of all retained runnings and has the same sign as the forward runnings.

Reconnaissance

Plans for proposed leveling should show the route plotted on maps of the area. Because available maps may not include all existing field conditions, a reconnaissance should be made to determine if the most feasible routes are being proposed. If alternate routes for some of the lines would better develop the net, the alternate routes should be followed unless prohibited by project requirements. In considering alternate routes, the proper spacing of lines must be maintained (within reasonable limits).

Permission should be obtained from local authorities if it is necessary to operate along railroad right of ways, on private land, roads, bridges, busy highways, or turnpikes. This should be done well in advance of the actual field work. When leveling along busy bighways, the Project Director should inform local law enforcement agents of the safety precautions that will be taken.

Connections to Existing Surveys

Minimum requirements for connections to existing vertical control surveys appear in table 7. Connections must verify observed elevation differences of establisbed bench marks within the prescribed limits of the lowest order survey of previous or new leveling; that is, a third-order survey must check between two marks of a previous first-order survey to third-order tolerance limits. Check connections shall be single-run unless an error is suspected in the new leveling. In first- and second-order check connections to previous first- and second-order surveys, NGS, or the originating agency for the previous survey, should be consulted if satisfactory checks are not

TABLE 7.—*Minimum requirements for connections to previous vertical control surveys*

New survey order \ Previous survey order	First	Second	Third
First	Check 3 marks. Contact originating agency after 4	Check 2 marks. Contact originating agency after 3	Check 2 marks
Second	Check 2 marks. Contact originating agency after 3	Check 2 marks. Contact originating agency after 3	Check 2 marks
Third	Check 2 marks	Check 2 marks	Check 2 marks

obtained after tying the survey to one mark greater than the number required for the check. (See table 7.) Failure to check previous leveling can be caused by crustal motion, physical disturbances, or geological phenomena. Checking the elevation difference between two bench marks located on the same structure, or so close together that both may have been affected by the same localized disturbing influence, is not considered a proper check. If the elevation difference between two consecutive bench marks does not check, but the sum of the two "new and old" differences agree within stated tolerance limits, it can be accepted as a "two bench mark" check ("jump" check). (For example, see Coast and Geodetic Survey *Special Publication* 239, Manual of geodetic leveling (Rappleye 1948b: 32–33).)

Equipment

Instruments

First-order leveling requires a level instrument with an optical micrometer. Beyond this requirement, any instrument that consistently yields results of the order and class of required leveling can be used in geodetic leveling. The properties of a geodetic level include ruggedness, precision in manufacture, and sufficient sensitivity to permit observations of the accuracy required by the specifications.

For compensator levels, the compensator substitutes for the sensitive spirit vial (tube) and tilting screw. The compensator should be sufficiently sensitive and consistent to set the line of sight within 0.″25 accuracy for first-order, or 0.″5 accuracy for second-order surveys. The compensator should not be oversensitive to vibrations caused by wind or passing vehicles.

The level vial of a spirit level should be of uniform curvature and be sufficiently sensitive and consistent to set the line of sight within 0.″25 for first-order surveys or 0.″5 for second-order surveys. For instruments that have the bubble centered by optical coincidence with a bubble image magnification of 2 or 3, a sensitivity of 8″ to 10″ per 2 mm is adequate. The level vial should be built into the instrument to shield the vial from direct sunlight. Sun shades or shields should always be used for higher-order leveling with spirit levels.

The telescope should have a large objective lens (50- to 70-mm diameter of the free aperture) to ensure brightness and good definition of image. The magnification power of the telescope may vary from 30 to 50 diameters. A lower power telescope provides a wider field of view for short-sight distances. The reticule should be designed with the markings sandwiched between glass plates to ensure a dust-free image. A reticule with a horizontal "V" should be used with optical micrometer instruments and line-graduated rods.

The leveling instrument must be stable. Stability is accomplished by having a stable leveling base, one that is relatively wide compared to the height of the instrument. Minimizing the area exposed to wind also increases stability. The tripod must be strong and rigid. Its legs should be of sufficient length to allow the observer to stand erect. The shoes of the tripod should be pointed and tapered to provide stability in most types of soil.

Leveling Rods

First- and second-order leveling surveys employ precision leveling rods with graduated scales on steel-nickel alloy (invar) or similar strips having small coefficients of expansion. (See table 5.) The rods must be calibrated by a reputable testing laboratory against a standard (traceable to the national standard of length) at least once each year when in use. Rods are graduated in either 1 cm, 0.5 cm, 0.01 ft, or 0.01 yd. The units of the level micrometer and rod must be compatible. Rod scales on the invar strips are usually in the form of black and white rectangles (checkerboard) or line marks. Rods with line markings are used with levels that have optical micrometers. Rods with rectangular markings are used with levels that require the observer to interpolate to tenths of a rod unit. The numbers which identify the scale divisions are on the rod frame itself.

Most modern precise rods have two parallel scales which differ by a constant amount (rod constant). Observing both scales can increase accuracy and facilitate the detection of reading and recording blunders. The two rods of a pair should have different rod constants to detect transposition of backsights and foresights. Rod frames are made of wood or metal and must have a spherical level vial attached, to aid in placing the rod in a vertical position. Invar strips fit into guides or grooves and are under constant tension. The invar strip and the rod frame must expand and contract independently.

Turning Points

Metal turning pins should be used whenever possible. They cannot be used in sandy or marshy ground, on concrete surfaces, or on highly packed consolidated gravel. Marshes or sandy surfaces should be avoided for first-order level lines. When such surfaces cannot be avoided, wooden stakes are to be driven to a firm depth and a double-headed nail driven into the top of the stake to provide a turning point. A heavy (at least 7 kg), well-designed turning plate (turtle) should be used on concrete or hard-packed surfaces. Extreme caution should always be exercised when using a turning plate.

Leveling Procedures

In addition to the actual observations (rod readings), additional information must be provided before survey data can be incorporated into the National Vertical Control Network. *NOAA Manual* NOS NGS 2, Input formats and specifications of the National Geodetic Survey data base, volume II: vertical control data (Pfeifer and Morrison 1980) specifies the required data and formats.

Observing procedures should be designed to minimize observation errors. If double-scale rods and compensator instruments with parallel plate micrometers are used, the recommended observing sequence is: backsight low scale; backsight stadia; foresight low scale; foresight stadia; reverse compensator or off-level and relevel the instrument to reposition the compensator; foresight high scale; and backsight high scale. The same observing sequence can be used with spirit levels with parallel plate micrometers without off-leveling the instrument.

This observing sequence reduces the effects of hysteresis (under- or over-compensation) and systematic vertical movement of the tripod or rod supports during observing. After completion of the setup, the rear rod is moved forward to become the forward rod for the next setup, and the forward rod remains in place to become the rear rod. This "leap frogging" procedure, when used with an even number of setups for each section, cancels rod index errors and reduces the accumulation of rod verticality errors.

In forward and backward leveling, two runnings are made of each section, one in the forward direction of the line and the other, at some other time, in the backward direction of the line. To prevent transpositions or misreadings, a single setup cannot be used for both a forward and a backward running of a section. The instrument height is to be changed by several centimeters between runnings. Reversing the sign of the backward running, and meaning the result with the forward running, reduces the effects of small systematic errors such as vertical movement of the forward rod support when the instrument and rear rod are moved forward for the next observations. Comparison of forward and backward section runnings permits detection of blunders. In general, efficient leveling operations are conducted with such care that less than 5 percent of the sections should require additional runnings to obtain specified checks between forward and backward runnings. When the amount of additional runnings exceeds 5 percent, field procedures and surveying equipment should be investigated. Exceptions include observations involving desert terrain, marshy ground, or other unusually difficult observing conditions.

In double-simultaneous leveling (Whalen and Balazs 1976), two elevation differences (one from the low and one from the high scale readings) are determined and compared at each instrument setup. This provides better checks than those obtainable by comparing low and high scale rod readings on individual rods. The two elevation differences obtained in this manner are not as independently determined as are forward and backward section runnings. The procedure should be used only if it can be evaluated with loop closures. Historically, double-simultaneous (also called double-rodded) leveling was accomplished using two turning points for each rod. The check was made by comparing the heights of instrument computed from elevations on the two back turning points. The back turning points from the setup were left in place until the check was obtained.

Double-simultaneous leveling is currently being performed with a single turning point for each rod, using double-scale rods. Low and high scale elevation differences are computed, differenced, and compared to a tolerance (table 6) at each instrument station. If the tolerance limit is exceeded, reobservations are made prior to moving the instrument and rods. When the two rods have different rod constants, the difference check detects transpositions of backsight and foresight readings. The mean of low and high scale elevation differences is calculated for each instrument station and the means are summed to obtain the elevation differences for the section. The direction of running should be reversed on alternate sections, or at least on alternate work days, to reduce accumulation of small systematic errors such as rod support movement along the level line.

The lengths of sight should provide clear and steady images of the rod. This allows proper settings with an optical micrometer (or a good interpolation of the wire readings). However, undue caution and very short sights reduce progress without appreciably increasing leveling accuracy. For favorable topography and weather conditions, the sight lengths should approach the maximum value. (See table 6.) The line of sight should never be nearer than 0.5 m to the ground at any point along either the backsight or foresight.

For levels with a critical focus (for example, Zeiss Ni-1), the maximum sight length unbalance at a setup shall not exceed 2 m for first-order, class I leveling. If possible, unbalanced sight lengths should be compensated by similarly unbalancing the sight lengths in the opposite direction during the following setup. When the collimation error of an instrument remains in adjustment with time and temperature changes, sight distances are carefully balanced, and the unbalance between setups does not exceed 5 m, weekly determinations of the level collimation error (by the Peg Test) are sufficient. However, if there is reason to believe the collimation of the instrument has changed, the Peg Test should be performed immediately. Also, all instruments recently acquired, repaired, or removed from storage should be tested before they are used on a survey. A weekly check of nonreversible compensator instruments shall be made to ensure the compensator is functioning properly. The method outlined in NOAA Form 77-81 can be used. Once a week, the adjustment of the spherical (plumbing) levels on the rods must be tested. For procedural guidelines, refer to U.S. Coast and Geodetic Survey *Special Publication* 239, Manual of geodetic leveling (Rappleye 1948b); and Lake Survey *Miscellaneous Paper* 69-4, Experimental techniques for levels of high precision using the Zeiss Ni-2 Automatic Level (Berry 1969).

Sources of Error in Leveling

Figure 4 shows a matrix of leveling error sources and the proper procedures to detect and control them (Whalen and Balazs 1976). These errors are of three types: blunders, systematic errors, and random errors.

Blunders

If the forward rod support is significantly disturbed as the rear rod and instrument are moved forward, a serious blunder may occur. If it is certain that the forward rod has been disturbed, the observer should be notified immediately and the section reobserved again from the bench mark. If light-weight turning plates are used, it may be difficult to determine if the plate was disturbed. The probability of such an occurrence is reduced if heavy (at least 7 kg) turning plates are used and set well into the surface. If sections are run in the forward and backward mode, the blunder may be detected when comparing backward and forward runnings. For double-simultaneous or single-run leveling, this blunder may be detected by comparing new section runnings with previous levelings, or by checking loop or line closures. Because this type of error is not easily detected in double-simultaneous leveling, additional precautions must be taken.

If double-scale rods are used with the recommended observing procedure, blunders in reading the rod graduations should be detected when comparing low and high scale elevation differences at each instrument station. If a blunder has occurred, the rods can be reobserved before the instrument and rear rod are moved. If double-scale rods are not used, or if the same reading error has been made on both scales, the blunder may be detected in forward- and backward-run leveling by comparing the forward and backward runnings. In this case, one or both of the section runnings may have to be repeated. If single-scale rods are used in single-run leveling, the blunder may be detected by comparing new leveling to previous leveling or by checking loop or line closures. If the blunder is within a few centimeters, it is not likely to be detected by loop or line closures.

Reversal of backsight and foresight rod readings during the recording process can be detected and corrected at the instrument station by comparing low and high scale elevation differences if the double-scale rods have

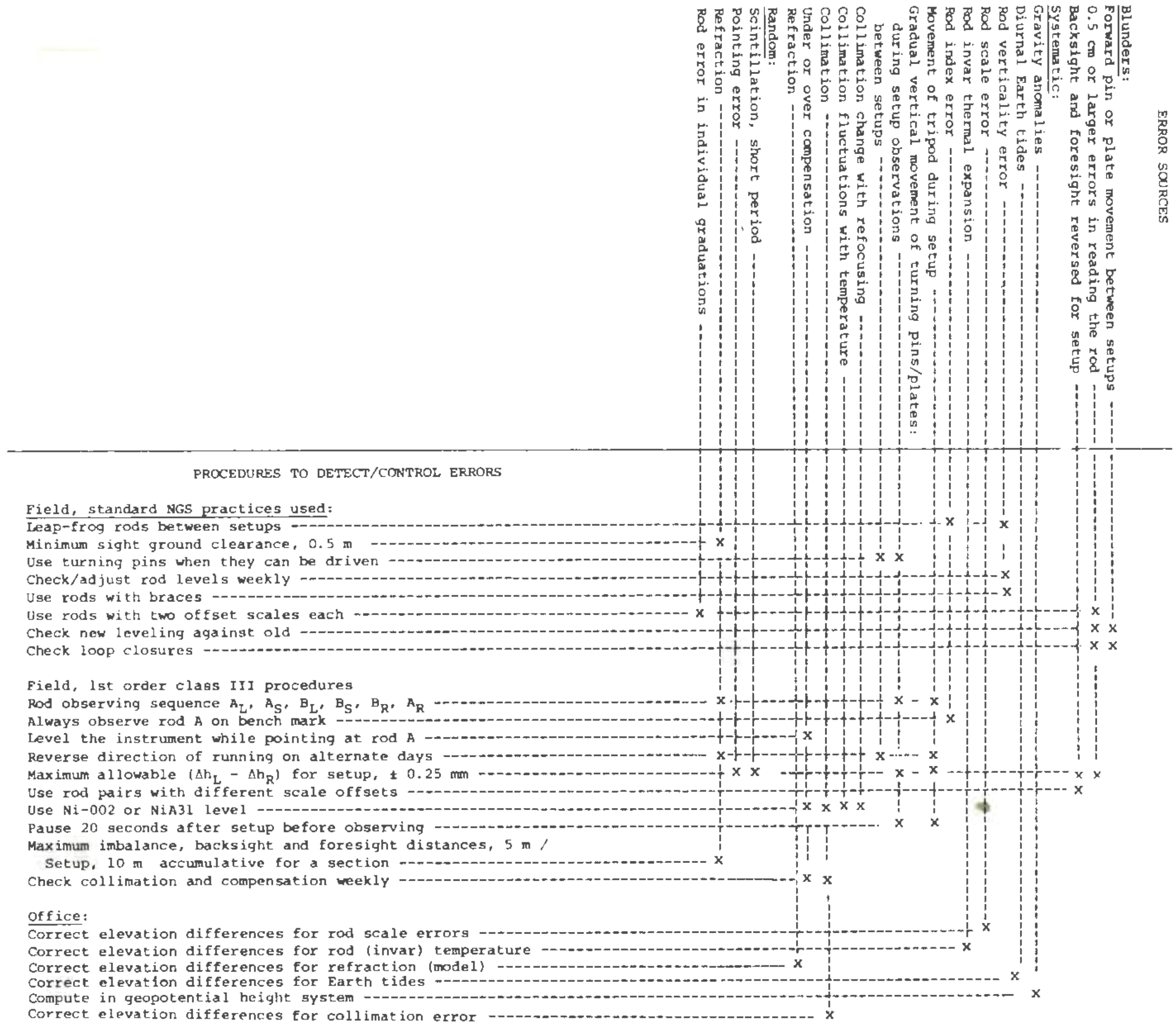

FIGURE 4.—Leveling error sources and procedures employed for detection and control.

different rod constants and the recommended observing sequence is used. The difference of the rod constants, with appropriate sign, must be applied to the high scale elevation difference before determining the check. If double-scale rods with the same constants are used, the blunder may be detected in forward- and backward-run leveling by comparing forward and backward section runnings. In double-simultaneous and single-run leveling, the blunder can be detected by comparing new leveling to the previous leveling or by checking loop and line closures. When leveling on a flat surface, the blunder may not be detected unless double-scale rods with different rod constants are used.

Systematic Errors

Discrepancies that result from gravity anomalies can be minimized by using observed or appropriate interpolated gravity values and then computing the results in the geopotential height system. Errors from astronomic effects can be reduced by applying appropriate astronomic corrections. The rod verticality error is reduced by using rod braces and a well adjusted spherical level attached to the rod. Rod scale and invar thermal expansion errors can be minimized by applying corrections determined from rod standardization results and observed temperatures. Using an even number of setups for each section and leapfrogging the rods cancel rod index errors. If leapfrogging or an even number of setups is not used, correcting for rod index errors or pairing rods with equal, or nearly equal, index errors reduces this error.

The effect of a gradual movement of the instrument tripod during an observation tends to cancel itself if double-scale rods are used with the recommended observing sequence, and low and high scale elevation differences are meaned at each instrument station. If double-scale rods or the recommended observing sequence is not used, the error is reduced by meaning forward and backward section runnings, or by reversing the direction of running on alternate sections or work days in double-simultaneous or single-run leveling.

Rod support movement during the observing process can be reduced by using turning pins, or wooden stakes, (when appropriate) or by using heavy turning plates set firmly on the ground. The error can also be reduced by using double-scale rods, with the recommended observing sequence, and meaning the resulting low and high scale elevation differences at each instrument station. The error may be large enough to cause the low versus high scale elevation difference check to fail and may necessitate a reobservation of the setup rod readings. If this procedure is not followed, the error can be reduced by meaning forward and backward section runnings, or by reversing the direction of double-simultaneous or single-run leveling on alternate sections or work days.

Closely balanced backsight and foresight distances at each instrument station reduce collimation error Instruments should be checked for collimation error and adjusted if necessary prior to use. They should then be rechecked at 1-day to 1-week intervals (depending on the individual stability of the adjustment). If an instrument is subjected to shock, it should be rechecked before the survey continues. To the extent that a collimation error remains constant from the time it is determined until leveling observations are made, runnings can be corrected for the collimation error. However, this is not often possible because collimation error tends to change with changes in temperature and time. For the Ni 002 instrument, collimation errors are minimized if they are symmetric about the quasi-absolute horizon in the two compensator positions and if the instrument is used as described. Except for the Ni 002, compensator instruments should be checked weekly for compensator error.

Refraction errors can be minimized by increasing the height of instrument and by reducing sight distances when refraction conditions are unfavorable (the effect of refraction is proportional to the square of the sight distance). The worst refraction conditions occur when the observer is observing near the top of one rod and the bottom of the other, with maximum allowable sight distances on sunny, windless days. Refraction does not usually cause serious leveling errors when observations are made over level ground (readings on the two rods are almost equal) and sight distances are balanced. The amount of refraction error is usually less on cloudy, windy days. The line of sight should never be nearer than 0.5 m to the ground along either the backsight or foresight. When two or more temperatures are observed at significantly different heights above the ground along the line of sight, refraction errors can be reduced by computing and applying refraction corrections. These errors can also be reduced by applying refraction corrections using observed or modeled temperature profiles.

Random Errors

Short-period scintillation ("boiling" of air) causes rod graduation images to appear unstable or blurred and increases pointing error. This error can be minimized by increasing the height of the instrument, reducing sight distances, or by meaning multiple micrometer readings and using the mean as the rod observation.

Long-period scintillation can cause an error of 1 mm or more in a single rod reading. This condition usually occurs when the temperature gradient is positive. (Air near the ground is cooler than the air above.) Positive temperature gradients usually occur at night, in the early morning, and in late afternoon. Minimizing this error requires that observations be taken only when the temperature gradient is negative. (Air layers close to the ground are warmer than the layers above.) If instruments are not available to measure temperature profiles, observations should be made during the time interval from 2½ hours after sunrise to ½ hour before sunset. The error may be detected by comparing forward and backward section runnings, or by comparing new to previous leveling in double-simultaneous or single-run leveling. The problem may also be detected by observing the reticule against a rod graduation for about 1 minute. If the rod graduation remains stable relative to the reticule, long-period scintillation is probably not present.

Random pointing or reading errors result from a combination of factors involving the instrument, observer, and sight length. This error depends on the ability of the observer to set the horizontal "V" of the reticule on a rod graduation using the micrometer, or to estimate the reading in three-wire or single-wire leveling. The error can be detected and reduced in the same manner as short-period scintillation errors.

Errors caused by inconsistencies in rod graduations can be reduced by using double-scale rods, observing two readings on each rod, and meaning the resulting low and high scale elevation differences for each instrument station. The errors can be minimized by calibrating each rod graduation and using the calibrated, instead of the nominal, values of the graduations.

Monumentation and Description

Permanent bench marks should be established at 1- to 3-km intervals. The average distance between bench marks along a line should be 1.6 km. If there are no bedrock outcrops or suitable stable structures (at least 5 years old) in which to set marks, rod-type bench marks should be used. Consult *NOAA Manual* NOS NGS 1, Geodetic bench marks (Floyd 1978) for detailed information. In determining the required density, bench marks that have been adequately set by other organizations should be used to comply with the density requirement. Other types of marks, such as chiseled squares, bolts, rivets, and unmarked points cannot be considered in the density requirement. In areas studied for rapid earth movement, it is good practice to space bench marks at 0.75 km or less along each line. A sufficient number of high quality marks should be set to define the areas of movement and to ensure the continuation of leveling records.

For first-order leveling, at locations where line junctions are planned or already exist, three highly stable bench marks should be established within a radius of about 1 km and spaced so all the marks will not be affected by the same local disturbance. A minimum of two highly stable bench marks should be set within a radius of about 1 km from the intersection of first- and second-order or two second-order lines. Junctions with third-order lines do not require multiple bench marks. Bench marks set in bedrock, deep rod marks anchored in a stable earth stratum or solid rock, or disks set in old massive structures with deep foundations should be used for first-order junctions. Tie-leveling to bedrock bench marks located as far as 8 km from a line junction is highly recommended. Bench marks set in stable buildings, or structures, or deep rod marks set in a stable earth stratum or solid rock, may be used for second-order junctions.

Except in unusual cases, permanent bench marks should be set several weeks before the first leveling of the line, to allow time for stabilization. If it is necessary to set new marks during the leveling survey, those set in bedrock or in older structures with foundations on bedrock are not likely to change significantly. Bench marks should be set in bedrock outcrops whenever possible. Sleeved deep-rod marks set in bedrock or stable strata are usually stable and can be leveled almost immediately. All new permanent bench marks must be marked by a metal tablet, secured into rock, brick, stone, concrete, or to the top of a metal rod or pipe. Exceptions include certain special deep marks, where the identification may be cast in a concrete or metal cover.

Whenever a bench mark is set, a witness sign should be set nearby to aid in future recoveries and to prevent unnecessary destruction of the mark during construction work. Omit the witness sign if it would be unsightly, pose a hazard to traffic, or is so requested by the property owner.

When a permanent bench mark is established or connected to the level net for the first time, a description report should be prepared. Recovery notes should be prepared for all recovered bench marks which have been previously described. For details on preparing descriptions and recovery notes, see publications by Pfeifer and Morrison (1980) and Whalen (1979). Descriptions and recovery information should be written with sufficient detail to locate the site of the bench mark easily, including the distance and direction from the witness sign. A sufficient number of reference points should be provided to facilitate bench mark recovery if physical changes occur at the site.

Gravity in Vertical Control Surveys

The equipotential surface, which is everywhere normal to gravity, having the same geopotential as mean sea level is called the geoid. Since the Earth is not homogeneous, the geoid is an undulating surface. An ellipsoid which approximates the geoid surface is used to compute gravity anomalies. Adequately spaced gravity stations can be used to estimate gravity at bench marks where actual gravity was not observed.

The level bubble (or the compensator in compensator levels) indicates the local direction of gravity, *not* the geodetic ellipsoid. Gravity is the resultant of gravitational attraction and the centrifugal force produced by the Earth's rotation. Gravitational attraction varies by a small amount over the Earth's surface, but centrifugal force varies from zero at the poles to a maximum at the Equator. Rotational forces are also different at different elevations. These cause level surfaces to be nonparallel.

Because level surfaces are not parallel, it is necessary to use one of two principal approaches when adjusting leveling nets that cover a large area. The first approach, used in the General Adjustment of 1929 (and currently used in the reduction of subsequent leveling to NGVD29), is to apply an "orthometric" correction to the north-south range of leveling lines to correct for the northward convergence of equipotential surfaces. Theoretical (normal) gravity at the mean latitudes of the level lines, or sections, is used for this method. This correction counteracts the nonparallelism of level surfaces that previously distorted elevations in the north-south direction. Orthometric corrections that are based on normal gravity are inexact because they do not take gravity anomalies into account. *Precise* orthometric height differences can be computed using *observed* gravity values.

A second procedure computes and adjusts leveling nets on the basis of geopotential differences. The geopotential difference between two points incorporates both observed gravity and the observed height difference. This procedure is more exact than the procedure that does not incorporate gravity anomalies. The resulting elevations in geopotential units can be converted to orthometric values by several methods. In order to compute in the gpu system, gravity (either observed or appropriately interpolated) must be known at bench marks to the tolerances given in table 6. Elevations in gpu are the same for a given point or level surface regardless of the leveling route. Therefore, adjusted elevations in geopotential units give the true relationship (hydrostatic head) between bench marks. Although scientists have recognized for many years that gpu are superior to the "orthometric" procedure which incorporates normal gravity, no effort was made to convert to the gpu system because the cost of determining actual gravity by pendulum instruments was prohibitive.

Relatively inexpensive gravity observations can now be made using modern gravity meters. In a systematic releveling and readjustment of the primary net of vertical control surveys, gravity anomalies will be incorporated into the final results for computations and adjustments in gpu. Orthometric elevations computed from gpu will also be published.

Doppler Satellite Surveys

Doppler satellite surveying is a method of determining positions of points on the Earth's surface by observing the Doppler shift of radio transmissions from satellites of the U.S. Navy Navigation Satellite System (NNSS). NNSS was developed for the Navy as a worldwide all-weather navigation system, and provides position fixes at time intervals of 2 hours or less. Observations of these satellites began in 1971, when portable tracking receivers first established precise positions on the Earth's surface. Since then, the application of Doppler satellite surveying techniques expanded, until today they are used worldwide (Defense Mapping Agency National Ocean Survey 1976, 1979).

Specifications in this section provide guidance in methodology for Doppler surveying activities. They are the result of extensive field tests performed by NOAA/NOS National Geodetic Survey, the Defense Mapping Agency (DMA), and the Geodetic Survey of Canada. The Doppler specifications, in particular the standards of accuracy for Doppler surveying are considered to be provisional until further tests are completed with various station spacings and network configurations, utilizing improved tracking equipment and refined computer programs for the reduction of data.

System Description

Five operational satellites currently are in circular polar orbit approximately 1000 km above the Earth's surface. Orbital periods are approximately 105 minutes. The Earth's rotation causes a satellite to cross the Equator on each revolution approximately 26° in longitude west of the previous crossing. Table 8 lists the NNSS satellites now in orbit. The satellites transmit two ultrastable co-

TABLE 8.—*Satellites of the U.S. Navy Navigational Satellite System in orbit as of May 1980*

Name	Catalog number	Satellite designation		
		APL[1]	International	NSWC[2]/DMA[3]
Oscar 11	10457	30110	1977 106A	93
Oscar 13	02807	30130	1967 48A	59
Oscar 14	02965	30140	1967 92A	60
Oscar 19	04507	30190	1970 67A	68
Oscar 20	06909	30200	1973 81A	77

[1]Applied Physics Laboratory, Johns Hopkins University, Laurel, Md.
[2]U.S. Naval Surface Weapons Center, Dahlgren Laboratory, Dahlgren, Va.
[3]Defense Mapping Agency, Washington, D.C.

herent frequencies, one at 150 MHz and the other at slightly less than 400 MHz.

A data stream is phase modulated on the 400-MHz carrier to provide both time and "broadcast" ephemeris for the satellite. The broadcast ephemeris, which describes the satellite's position in space, is a predicted orbit based on Doppler observations previously acquired by four tracking stations located in the United States (Maine, Minnesota, California, and Hawaii).

Doppler satellite observations, reduced using the broadcast ephemeris, yield point positions with sufficient accuracy to satisfy many cartographic or similar control requirements. The more accurate "precise" ephemeris, which is generated for selected satellites from Doppler observations acquired by a tracking network (called TRANET) composed of 15 to 20 stations, radically improves the positioning accuracies for single station observations. The precise ephemeris represents a coordinate system defined as Naval Surface Weapons Center 9Z-2 (NSWC 9Z-2), which can be related to the World Geodetic System (WGS-72) and many local geodetic systems. The broadcast ephemeris represents a similar but not identical geocentric system with additional uncertainties of several meters in each coordinate. Precise ephemerides are computed by DMA to support geodetic applications.

Modes of Operation and Expected Accuracies

Points on the Earth's surface can be positioned by various modes (point positioning, simultaneous point positioning, translocation, semi-short arc or short arc) using either the broadcast ephemeris or the precise ephemeris (Defense Mapping Agency 1975).

Multiple passes collected with a single Doppler receiver in the point-positioning mode are used with an ephemeris to determine an independent station position in geocentric coordinates (X,Y,Z) referenced to the Earth-centered satellite coordinate system. The geocentric coordinates can be expressed in geodetic coordinates (latitude, longitude, and height above ellipsoid). Geodetic coordinates from precise ephemeris station positions, defined by the NSWC 9Z-2 station coordinate system and the NSWC 10E-2 gravitational model, are computed on the NWL 8E ellipsoid ($a = 6378145$ ms, $1/f = 298.25$). Doppler positions determined with the precise ephemeris can be directly transformed to the WGS-72 datum.

When employing the point-positioning mode as the Doppler surveying technique, the user may wish to transform the Doppler satellite-derived geocentric coordinates to the local geodetic system. The process of deriving the coordinate shifts and transforming the Doppler position requires a thorough understanding of datum transformation concepts and procedures. Local geodetic control biases must also be known.

When the relationship between WGS-72 and a local geodetic system is not known, or if the Doppler positions were determined using the broadcast ephemeris, the transformation parameters must be determined. Occupation of a station with known local geodetic coordinates allows the transformation parameters to be derived for subsequent use with Doppler positions that were not established on a station tied to the local geodetic system. In consideration of local geodetic control biases, the transformation parameters should be used only in the near vicinity of the arc or in relation to other existing stations in the same arc. Examples of datum transformation are given in *Defense Mapping Agency Technical Manual* T-3-52320, Satellite records manual-Doppler geodetic point positioning (Defense Mapping Agency 1976: C1-C7).

Stations are simultaneously occupied in figures of two or more in the simultaneous point-positioning mode. The data are independently reduced as in the point-positioning mode and differenced to form relative positions. The simultaneous observations are performed during a common time period, but do not necessarily include common satellite passes.

The translocation or short-arc modes of Doppler surveys can be employed to obtain very accurate relative positions even if the precise ephemerides are not available. In the translocation mode, observations are simultaneously collected, usually at two stations. Statistical correlation performed during data reduction improves the accuracy of relative positioning when the broadcast ephemeris is used. The principal error sources affecting an individual satellite position fix are the ephemeris errors and the residual tropospheric and ionospheric refraction errors. Improved compensation for these errors is possible when the same signal is received at separate sites. The maximum spacing between sites is generally limited to approximately 500 km (or less if compatability with existing control is to be maintained) so that desirable portions of satellite passes can be tracked simultaneously. During processing, enforcement of simultaneity of data points is optional. When simultaneity is enforced, it is generally referred to as rigorous translocation.

The short-arc and semishort-arc modes allow for small adjustments in the orbit instead of holding the satellite ephemeris fixed, as is done for the translocation method. The translocation method assumes that orbit errors affect positioning of all sites in the same way, whereas the short-arc technique adjusts the reference orbit while simultaneously solving for positions. The a priori ephemeris is given six adjustable parameters in short-arc processing. If the a priori ephemeris is given one to five adjustable parameters, the method becomes semishort-arc processing.

For relative positioning, which in many ways is similar to trilateration, the positions of new stations are determined using interstation distances in a combined adjustment, where the base station positions are generally held fixed. These fixed stations usually have a known position on a given geodetic datum. While data are being collected at the fixed station(s), one or more additional receivers are circulated among the various unknown stations comprising the net. The data collected simultaneously are subjected to postprocessing to determine a position relative to the base stations. Using several different approaches, all stations become interrelated through connections to the base stations. The positions of the base stations can be referred to the local geodetic datum. They can be also be established by point positioning or by extension of a relative positioning method.

Although point positioning is the least accurate mode to use when only the broadcast ephemeris is available, this technique can be performed with a single receiver and fairly simple computations. Translocation and short-arc techniques are the most accurate modes when the precise ephemeris is not available, but for these modes field and computational procedures are more complex.

Because all satellites are capable of providing the broadcast ephemeris, it may be more economical to use the broadcast ephemeris rather than the precise ephemeris to meet control requirements. This can reduce significantly the period of occupation for a station. Data processing can also be performed more quickly because no time is lost waiting for posttracking orbital data in order to generate the precise ephemerides.

It may be necessary to adjust Doppler survey measurements for scale correction when the highest order accuracy is required. This correction is dependent on the method of data reduction and the type of ephemerides used. For distances less than 100 km, the effect of the scale correction should not exceed 10 cm for the broadcast ephemeris. For precise ephemerides the correction is much smaller and thus can be ignored.

Table 9 lists the estimated accuracies (1 sigma, 68-percent confidence region) attainable with Doppler surveys. It is important to note these estimates are considered representative values only. Geocentric position accuracies are stated for each coordinate component (X,Y,Z). Relative position accuracies are for the length of the vector base line (B) between the observing stations. The accuracy estimates are primarily based on the comparison of Doppler survey results to similar values obtained from high-precision traverses, first-order triangulation, and very long base line interferometry.

Investigations are in progress to determine if improved tracking equipment, and more refined data processing and reduction programs will yield improved accuracies for Doppler satellite surveys.

Because many factors affect the accuracy of positioning by Doppler techniques, the operation of Doppler receivers and the computer reduction programs should be checked by testing the precision of the measurements and by comparing the results with known geodetic control. These calibration surveys will serve to demonstrate, with reasonable assurance, that the required accuracy for a survey can be met.

Specifications

Tracking Equipment

Doppler geodetic tracking receivers are available from major Doppler equipment manufacturers. All receivers are compact and portable, and are not affected by weather conditions. The weight of a system ranges from 30 to 50 kg. A system consists of a receiver, recording device,

TABLE 9.—*Accuracy estimates attainable in 1979 for Doppler satellite surveys*

Reduction method	Satellite ephemeris	Number of passes[1]	Approximate number of observation days[2]	Estimated accuracies[3] Geocentric (σ_X, σ_Y, σ_Z) cm	Relative positions (σ_B) cm
Point-positioning	Precise	40	7[4]	70	50[5]
Simultaneous point-positioning	Precise	80	14[4]	50	35[5]
Translocation	Broadcast/ precise	40	3[6]	—	35[7]
Short-arc	Broadcast/ precise	30	2[6]	—	30[7]

[1]Representative of the number of passes necessary to achieve stated accuracy estimates; more or fewer passes may give different results.

[2]Based on the number of passes available on a typical day at latitudes ranging from 25°–45°.

[3]Rough estimates of possible accuracies (1σ). The values will vary, depending on data quality. Data quality is related to instrumental errors, local effects, and degree of refinement of computer programs used for reduction. In addition, for relative positioning the accuracy estimates depend on the number of receivers available and network design.

[4]When using only precise ephemerides, there are usually only two satellites available; six to seven passes per day.

[5]Relative position uncertainties for station spacings of 200 to 4000 km.

[6]When using only broadcast ephemerides, as many as five satellites are now available; 14 to 20 passes per day.

[7]Relative position uncertainties for distances to about 200 km between stations. Station spacings of 200-500 km will result in larger errors.

antenna/preamplifier, and cables. Some have a microprocessor. The power requirements range from about 10 to 125 watts.

The reference frequency for the receivers is usually built into the system as 5×10^{-11} parts per 100 seconds (or better). The frequency drift may be periodically monitored during field operations to verify frequency stability. Atomic frequency standards with short-term stability of better than 5×10^{-12} parts per 100 seconds can be used as an auxiliary reference frequency source. To ensure the collection of high-quality data, crystal oscillators must either be connected continuously to a power source during transport between stations or be powered for a minimum period (typically 1 to 3 days) before initiating data collection. For atomic frequency standards less warm-up time (for stabilization) is required before the observations are begun.

Site Selection

As a general rule, the two primary requirements for successful operation at a Doppler survey station are a relatively unobstructed horizon and the absence of radio interference. Additional factors which must be considered when selecting a site are security, local survey requirements, and electrical power requirements.

Signal reception. The satellite signal is an electromagnetic wave which may be absorbed or refracted if physical objects block the ray path between the satellite and the ground receiver. Therefore, the antenna must be located where there are no obstructions above 10° of the horizon. If possible, the antenna should be positioned at normal tripod height. Under special circumstances, it may be necessary to elevate the antenna above trees or other obstructions by mounting it on a mast. Successful observations are possible by using such a mast, provided the antenna ground plane is effective in minimizing multipath problems. The antenna can be placed on a flat-roofed building if it is placed at least 2 m from the building edge. The antenna cannot be located near metal structures because reflected signals may cause the reception of two signals whose phase relationship would produce poor quality data.

Interfering signals. The antenna should be located where minimum radio interference occurs; near the frequency range of 150 and 400 MHz. Medium frequency radar and high power transmitting antennas must be avoided because they may emit sufficient radio energy to damage receiving equipment. Likewise, their harmonics may interfere with the 150 MHz or 400 MHz signals. High voltage power lines and transformers, or excessive noise from automotive ignition systems, must also be avoided. Interference is a function of the interfering frequency and its signal strength.

Antenna environment. Doppler receiver antennas are generally designed to withstand exposure to the elements. Antennas are sealed to protect them from moisture. They can operate at temperatures ranging from less than 0°F to more than +100°F (−18° to +38°C).

Receiver location. Because it is necessary to interconnect the receiver and the antenna/preamplifier, the receiver must be located within 30 m of the antenna (the normal cable length provided by the manufacturer). A longer but heavier cable (up to 60 m) may be used. Although newer receivers with self-contained recording devises are built to be durable, waterproof, and dustproof, protection from animals and certain environmental conditions (extreme temperatures, winds, or other severe weather conditions) should be provided.

Survey requirements. The basic survey requirements to be considered when selecting a Doppler survey site are:

1. Each Doppler station shall be monumented or marked. The station monument shall contain a disk stamped with the name of the establishing agency, the station name or number, and the year in which the station was established. (Doppler stations that will be incorporated into the NGS national network must have a numbered station designation; this is assigned by the agency responsible for incorporating the Doppler station positional information into the network.)

2. The antenna shall be located so it is directly above the station mark.

3. At least two reference marks, preferably three, shall be located within 30 m of the station monument. Distances, horizontal angles, and differences in elevations must be measured to facilitate future recovery and verification of the Doppler station.

4. If local ties are to be made to the Doppler station, it may be necessary to establish an additional reference mark for use as an azimuth mark. Depending on accuracy requirements, the azimuth can be determined from astronomic or sun azimuth observations or by using a north-seeking gyro-theodolite. A less accurate azimuth may also be determined by performing simultaneous Doppler observations between the azimuth mark and the station, and then reducing the data by a relative positioning method.

5. Depending on the application and required precision of the Doppler surveys, deflection of the vertical components may be required for the primary stations.

6. A precise elevation referred to the National Geodetic Vertical Datum of 1929 (NGVD29) shall be determined for all first-order or primary Doppler stations, including the base stations of a relative net of Doppler stations.

7. If a tie is to be made to local geodetic control, the Doppler station monument must be located so local survey marks are intervisible.

8. To facilitate recovery or to provide control for aerial photography, it may be necessary to photoidentify the station directly on available photography. If the station cannot be photoidentified directly, three photoidentifiable objects, at least 200 m apart, should be selected in the vicinity of the station. A survey tie shall be performed between these objects and the Doppler station.

Additional information on recommended specifications for monumentation, description, and vertical and horizontal ties to local control appears in the sections on Horizontal Control Surveys (p. 22) and Vertical Control Surveys (p. 34).

Power Requirements. Receivers that consume about 100 watts require alternating current from a commercial source or a small generator. Receivers with low power requirements (5 to 10 watts) can operate from a variety of rechargeable battery systems. Nearly all modern systems can operate for a period of 48 hours or more using a standard 80 ampere-hour, 12-volt battery.

Choice of Base Stations

At least one base station of a relative net of Doppler stations shall be occupied simultaneously when one or more new stations are being established. The base station must be an astronomic or primary geodetic station and have a precise elevation (which is referred to the NGVD). The base station shall be within 500 km of the new station(s) when data are reduced by the translocation or short-arc methods. The network can be strengthened by an interlocking pattern of simultaneous observations at three or more stations.

Spacing of Stations

The spacing of a network of Doppler stations determines the order of survey produced by the observations. Table 10 shows the minimum spacing between Doppler stations. Values are based on 1-sigma Doppler relative positioning accuracies (σ_B) given in table 9 and on the following formula:

$$\text{Minimum Distance} = \frac{2.00 \text{ Doppler relative positional accuracy } (\sigma_B)}{\text{Allowable relative positional accuracy (expressed as a ratio)}}$$

This formula is based on using a 95-percent confidence region in terms of a linear error. The value for σ_B is the uncertainty at the 68-percent confidence level in determining the base line vector between two stations. The 95-percent confidence region is approximately two times larger than that of the linear standard error.

TABLE 10.—*Minimum Doppler station spacing*

Order and class of survey	Allowable error (2σ or 95% confidence level)	Doppler: Relative positioning accuracies (σ_B) Minimum station spacing (km) 50 cm	35 cm	30 cm
—	1:250,000	250	175	150
First-order	1:100,000	100	70	60
Second-order, class I	1:50,000	50	35	30
Second-order, class II	1:20,000	20	14	12
Third-order, class I	1:10,000	10	7	6
Third-order, class II	1:5,000	5	4	3

The allowable errors include those stated herein and in *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1974) for primary and principal stations of the National Networks of Geodetic Control. Doppler relative positioning accuracies are in relation to stations positioned to similar specifications.

Doppler Satellite Receiver Operating Instructions

The satellite receiver is to be operated in accordance with the manufacturer's technical manual. All system checks should be performed in accordance with the specifications in the technical manual to ensure all data are of the highest quality.

All acceptable receivers have a satellite signal simulator available for checking the circuitry of the receiving system. Many receivers incorporate a built-in self-test feature while others require a separate signal simulator. The circuit check shall be performed when beginning observations at a station and, thereafter, as often as required by the manufacturer's specifications.

A record of the clock error can be maintained for some receivers. This is useful in verifying that the system is functioning properly. Other system checks may be performed including a frequency difference test, which compares the accumulated 2-minute Doppler counts for 150 MHz and 400 MHz transmissions. The frequency stability of the receiver reference frequency shall be monitored periodically by comparison with other standards or by maintaining a record of the drift relative to the satellite frequency.

A microprocessor is built into the newer Doppler receivers. It controls the receiver and performs data formatting, data verification, and operator-specified pass selection. An on-site data processing capability permits processing of satellite passes in a sequential point-positioning adjustment using the broadcast ephemeris. Solution diagnostics include statistical information on the position results, measurements of timing information, and reference frequency drift. These receivers also feature modular construction and, for some, built-in test equipment. The microprocessor and test equipment enable the operator to check the system's function and to locate faulty modules or circuits.

Antenna Installation

The antenna shall be centered over the station marker to the nearest 1 mm. The height difference between the station mark and the antenna's phase center (the reference point defined by the manufacturer to which the Doppler observations are referred) must be measured and recorded to the nearest 1 mm. If the phase center is not marked on the antenna, the bottom of the base shall be taken as the reference point. The reference point for the height measurements must be clearly described.

The antenna shall not be moved during station occupation. If for any reason the antenna is moved from its original position, it must be replaced so the phase center of the antenna lies within 1 mm of its original position.

If an antenna is moved while a pass is in progress, that pass is not usable. Antennas in exposed locations must be adequately secured to prevent movement caused by wind- or water-induced erosion.

Selection and Number of Satellite Passes

Satellite predictions. Satellite predictions may be generated to provide the following information for given dates and locations: satellite identification, time of rise, time of set, time of closest approach, rise azimuth, and elevation at closest approach. These predictions are used by the operator to select preferred satellites and to determine the observing period necessary to collect a specified number of passes. With a microprocessor and the broadcast ephemeris, these predictions can be generated in the field.

Conflicting passes. When satellite passes are in conflict (that is, when two or more satellites are in receiving range at the same time), action must be taken to record the most suitable pass. The following factors should be considered:

1. When reducing data with precise ephemerides, a satellite whose precise ephemeris is available has priority.

2. Satellite passes rising to 8° or less above the horizon at closest approach are generally unacceptable and should be rejected.

3. Observing satellites at more than 80° above the horizon may result in loss of signal at the highest part of the trajectory but may otherwise be acceptable.

4. When acceptable passes are in conflict, the passes that produce the most data should be selected.

Minimum number of passes and observing days. A basic requirement for a successful station occupation is a relatively even distribution of passes east and west of the station meridian. There should also be a relatively even distribution of north-to-south and south-to-north passes of the polar orbiting satellites.

The minimum number of usable passes required for each station depends on the data reduction method to be applied, satellite ephemerides used, and the accuracy requirements. Table 9 (p. 38) summarizes the pass requirements and includes the approximate number of observation days.

A usable satellite pass rises above 8° elevation at closest approach. A minimum of four complete 2-minute broadcast messages is required for broadcast ephemeris reduction. When reducing the passes with the precise ephemeris, a minimum of 3 minutes of data is required for the pass to be considered acceptable.

Meteorological Observations

When the highest possible accuracy is desired, measurements of atmospheric temperature, station pressure, and relative humidity shall be recorded for each satellite pass. Accurate corrections can be determined from the meteorological observations for systematic errors attributable to tropospheric refraction.

The temperature and relative humidity measurements should be collected, if possible, at or near the height of the phase center of the antenna. Except at base stations, requirements for meteorological data may be relaxed in a relative net if distances from the base stations are less than 100 km.

Observations of wet-bulb and dry-bulb temperature readings must be recorded to the nearest 0.5°C. Barometric readings (station site pressure) must be recorded to the nearest 1.0 millibar (mb) and if significant they must be corrected for difference in height between the antenna and barometer.

During automatic acquisition of Doppler data, continuous weather recording instruments may be used to collect meteorological observations.

Three-Dimensional Surveying

Several new, three-dimensional control surveying techniques promise increased accuracy and lower costs. Some of these systems are now operational, while others are still under development.

Extraterrestrial Geodetic Survey Systems

Extraterrestrial geodetic survey systems determine geodetic heights above the reference ellipsoid (not above the geoid, which is the reference for the National Vertical Control Network.) Thus these systems will not be a substitute for conventional vertical control procedures until the geoid is determined to sufficient accuracy to relate the two reference systems. Space systems *can* provide valuable information on changes in heights for geodynamics studies.

Global Positioning System

The NAVSTAR Global Positioning System (GPS) should impact greatly on future surveying technology. At present six GPS satellites are in operation. When fully operational in the late 1980's, the system will employ a total of 18 satellites. To ensure constant coverage, six satellites will be equally spaced in each of three orbital planes at an inclination of 63°. At least four satellites will be in view at any terrestrial location as the satellites revolve in circular 20,000-km orbits (12-hour periods). Ranges (distances) to any three of the satellites, computed by means of the travel time of the signals from the satellite to the ground, will provide the position of the ground receiver. Observations to a fourth satellite may be required to provide sufficiently accurate travel time to synchronize the ground station clock with the satellite.

The accuracy of an observed instantaneous geocentric position is expected to be better than 10 m. The relative position of two stationary receivers, if separated by less than 1000 km, should be determined to a 1-meter level of accuracy with only a few minutes of observing. Relative positioning to better than 10 cm may be possible with a single day's observations. The global positioning system holds great promise of providing the surveyor with the means of determining sufficiently accurate horizontal positions at a relatively low cost.

Very Long Base Line Interferometry

Another technique that will probably impact significantly on future surveying methods is Very Long Base Interferometry (VLBI). VLBI measurements can be used to estimate the vector separation between radio telescopes with an accuracy of a few centimeters or less, even when the telescopes are separated by transcontinental distances. Recent VLBI surveys have used large, fixed radio telescopes, although some work has also been done with portable instruments. Recent studies indicate it should be possible to use much smaller, less expensive, portable antennas if high strength signals from artificial satellites (for example, GPS satellites) are used instead of much weaker natural radio signals.

The high precision possible from VLBI measurements should lead to meaningful results in studies of crustal motion and deformation. This could have interesting implications for earthquake prediction research and related fields such as solid-earth geophysics.

High-Precision Photogrammetric Surveying

Specifications are now being developed for horizontal positioning by means of High-Precision Photogrammetric Surveying (HPPS) techniques. Numerous tests and evaluations of simulated and field observations show that accuracies at the 4- to 5-cm level (1 sigma) are possible in areas encompassing 400 square miles (1036 km^2) or larger. To achieve this accuracy, first-order control at 10- to 15-km spacing is required around the perimeter. Special paneling is used in conjunction with high precision photogrammetric instrumentation and procedures. An additional requirement is that the premarking pattern be spaced at about 1.5 km throughout the survey area. When the final specifications are issued, many surveyors are expected to densify survey networks by the HPPS method. Compared to conventional methods, it appears significant savings can be achieved by HPPS for areas having sufficient ground control.

Inertial Positioning System

A possible breakthrough in achieving a total surveying instrumentation package is the Inertial Positioning System (IPS). This system can determine astronomic azimuths, horizontal positions, heights, deflections of the vertical, and gravity values. In addition to being an all-weather, day-or-night system, it is also independent of line of sight. Although the instrumentation is complex, it is compact enough to be placed in a jeep or van-type vehicle or helicopter. At least one commercial firm and three geodetic organizations are employing IPS to establish control surveys. Over short distances (3–5 km) between high-order control, horizontal closures of the order of a few centimeters have been obtained in one instance. The original cost of the equipment is large; however, if the user considers the number of stations that can be positioned in a single day, the cost per station becomes very low when compared to those established by conventional survey practices.

Marine Geodesy

National programs for the development of food and mineral resources in the continental shelf and the Great Lakes area, plus emphasis on broader oceanic research, have created the need for special geodetic support. Specifically, geodetic support is required for:

- Position fixing of points along seaward boundaries (between the United States and international water).
- Markers or special instruments on the sea floor.
- Ships at sea performing various geophysical measurements.
- Relating the geographic positions of remote islands to continental datums and to a worldwide Earth-centered system.
- Precise determination of ocean heights for tidal research.
- More accurate geoid determinations.

These requirements emphasize the global characteristics of geodesy. The extension of geodetic control into the sea has become a major national task. Only a few of the classical geodetic surveying techniques used in the past to establish geodetic control on land are suitable in the sea environment. Therefore, Federal agencies and special institutes are developing new technology to meet these special needs.

Reliable electronic systems are available for offshore hydrographic surveying. If distances from shore do not exceed 120 km, positions can easily be determined with accuracies of 5 to 10 m. These surveys are classified as third order. For bathymetric mapping, such accuracies may be adequate, but for some boundary purposes higher precision may be needed. Various techniques (some modified from land-based methods) have been used with reasonable success to establish offshore control or geodetic ranges.

For position fixing in the deeper oceanic regions, the Loran C system and the Navy navigational satellite system are available. Unfortunately, neither system fully satisfies the instantaneous velocity requirements associated with measurement of gravity at sea. A program of continuing research—in which position, velocity, and gravity are considered as interrelated physical measurements—is essential.

A Doppler satellite positioning device can determine locations on a worldwide system to a meter or less in each component. However, the observing program takes several days, and a reasonably stable structure is required. In a few years, the NAVSTAR Global Positioning System will be fully operational, and it will probably be possible to determine positions accurate to less than 10 m almost instantaneously.

For the immediate future, it is not feasible to prepare standards and specifications for marine geodesy. However, it is essential that the nature of the tasks be described and the eventual need for standards be recognized.

Published Geodetic Data

Geodetic control data and cartographic information that pertain to the National Networks of Geodetic Control are widely distributed by the NGS National Geodetic Information Center (NGIC) to scientific and surveying-mapping communities. These include Federal, State, and local agencies; universities, private companies, and individuals. Data are furnished in response to individual orders, or by the automatic mailing list service (the mechanism whereby users who maintain active geodetic files automatically receive newly published data for specified areas)..

Geodetic control data for the national networks are primarily published as standard quadrangles of 30′ in latitude by 30′ in longitude. However, in congested areas, the standard quadrangles are 15′ in latitude by 15′ in longitude. In most areas of Alaska, because of the sparseness of control, quadrangle units are 1° in latitude by 1° in longitude. Data are now available in these formats for approximately 85 percent of the Nation, with the remaining 15 percent presented in old publication formats; i.e., State level lines, plane coordinate sheets, geographic position lists, and description booklets. Until the old format data have been converted to the standard quadrangle formats, the vertical and horizontal control data in the unconverted areas will be available only by complete county coverage. Field data and recently adjusted projects with data in manuscript form are available from NGS upon special request. The National Networks of Geodetic Control are cartographically depicted on approximately 850 different control diagrams. As an additional service, NOAA provides other related geodetic data; e.g., calibration base line data, gravity values, astronomic positions, preliminary adjusted horizontal positions, horizontal and vertical data for crustal movement studies, UTM coordinate data, and other information.

The NGIC receives data from all NOAA field operations and mark recovery programs. In addition, other Federal, State, and local governments, and private organizations contribute survey data from their field operations. These are incorporated into the NGIC control files. NOAA has entered into formal agreements with several Government agencies whereby NGIC publishes, maintains, and distributes geodetic data received from these organizations. Guidelines and formats have been established to standarize the data for processing and inclusion into the National Geodetic Survey data base. These formats are available to organizations interested in participating in the transfer of their files to NOAA.

Upon completion of the geodetic data base management system, publications generated from the data base will be automatically revised. A new data output format has been designed for both horizontal and vertical control information. These formats, which were necessitated by the requirements of the new adjustments of the horizontal and vertical geodetic networks, will be more comprehensive than the present versions.

New micropublishing techniques are being introduced in the form of computer-generated microforms. Some geodetic data are available on magnetic tape, microfilm, and microfiche. These services will be expanded as the automated publication system is fully implemented. Charges for automated data are determined on the basis of the individual requests, and reflect processing time, materials, and postage.

For additional information, contact

Director, National Geodetic Information Center
National Oceanic and Atmospheric Administration
Rockville, MD 20852

(telephone: 301-443-8631)

Maintenance of Survey Marks

Repairing and replacing existing survey marks play an important role in maintaining strong geodetic control networks. Organizations responsible for establishing survey marks should continually strive to keep them in good condition and update obsolete descriptions. This benefits not only the originator but all users.

The National Networks of Geodetic Control are serviced by NGS field personnel who have responsibility for specific geographic regions. One of their primary missions is to prevent destruction or damage to these monuments by educating the public. They are also responsible for repairing damaged monuments or relocating endangered marks to more secure sites. Often descriptions must be rewritten to reflect more accurately the locations and conditions of monuments. Witness signs are emplaced to protect the marks when possible.

The National Geodetic Survey urges all organizations to participate actively in the mark maintenance program. If you are aware of a survey mark that is endangered, please contact the originating organization. To report an endangered NGS mark, please call: 301-443-8319. (NGS will accept collect calls for information on endangered marks.)

References

Adams, O. S., 1915: Application of the theory of least squares to the adjustment of triangulation. *Special Publication* 28, U.S. Coast and Geodetic Survey, Washington, D.C., 200 pp.

Baker, L. S., 1968: Specifications for horizontal marks. *Technical Memorandum C&GSTM*-4, Environmental Science Services Administration (now NOAA, Rockville, Md.) 14 pp.

Berry, R. M., 1969: Experimental techniques for levels of high precision using the Zeiss Ni-2 automatic level. *U.S. Lake Survey Miscellaneous Paper* 69-4, U.S. Lake Survey (now NOAA/NOS, Rockville, Md.) 26 pp.

Best, A. C., 1935: Transfer of heat and momentum in the lowest layers of the atmosphere. Air Ministry, Meteorological Office, *Geophysical Memoirs* No. 65-8 (VII), His Majesty's Stationery Office, London.

Bureau of the Budget, 1967: Coordination of surveying and mapping activities. *Circular* No. A-16, Revised, May 6, 3 pp. Executive Office of the President, Bureau of the Budget (now Office of Management and Budget), Washington, D.C. 20503.

Bureau of the Budget, 1958: Programing and coordination of geodetic control surveys. *Transmittal Memorandum* No. 2, 1 p., and Exhibit C of Circular No. A-16, 4 pp. Executive Office of the President, Bureau of the Budget (now Office of Management and Budget), Washington, D.C. 20503.

Bureau of the Budget, 1953: Programing and coordination of surveying and mapping, *Circular* No. A-16, January 16, 15 pp. Executive Office of the President, Bureau of the Budget (now Office of Management and Budget), Washington, D.C. 20503.

Defense Mapping Agency, 1976: Satellite records manual—Doppler geodetic point positioning—data documentation and applications. *Defense Mapping Agency Technical Manual*, TM-T-3-52320, November, 47 pp.

Defense Mapping Agency, 1975: Field operations manual—Doppler geodetic point positioning. *Defense Mapping Agency Technical Manual* TM-T-2-52220, April, 76 pp.

Defense Mapping Agency/National Ocean Survey, 1979: *Proceedings of the Second International Geodetic Symposium on Satellite Doppler Positioning*, vols. 1 and 2, 1,333 pp. University of Texas at Austin, Tex., January 22-26. (Available from Dr. Arnold J. Tucker, 10000 Burnet Road, Austin, TX 78758.)

Defense Mapping Agency/National Ocean Survey, 1976: *Proceedings of the International Geodetic Symposium on satellite Doppler positioning*, vols. 1 and 2, 901 pp. University of New Mexico, Las Cruces, New Mex., October 12-14. (Available from Physical Science Laboratory, P.O. Box 3548, New Mexico State University, Las Cruces, NM 88003.)

Dracup, J. F., 1976, rev. 1979: National Geodetic Survey data: availability, explanation, and application. *NOAA Technical Memorandum* NOS NGS-5. National Oceanic and Atmospheric Administration, Rockville, Md., 41 pp.

Dracup, J. F., Fronczek, C. J., and Tomlinson, R. W., 1977: Establishment of calibration base lines. *NOAA Technical Memorandum* NOS NGS-8, National Oceanic and Atmospheric Administration. Rockville, Md., 22 pp.

Federal Geodetic Control Committee, 1974: *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys*. National Oceanic and Atmospheric Administration, 14 pp.

Federal Geodetic Control Committee, 1972: Exchange of Information. *Circular* No. 1 Federal Geodetic Control Committee, October 16, 6 pp. U.S. Department of Commerce, NOAA/NOS National Geodetic Survey, Rockville, Md.

Federal Register, 1976: *Federal Register*, vol. 51, no. 96, May 17, p. 20202. Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.

Floyd, R. P., 1978: Geodetic bench marks. *NOAA Manual* NOS NGS 1, National Oceanic and Atmospheric Administration, Rockville, Md., 56 pp. (GPO stock no. 003-017-00442-2).

Fronczek, C. J., 1977: Use of calibration base lines. *NOAA Technical Memorandum* NOS NGS-10, National Oceanic and Atmospheric Administration, Rockville, Md., 38 pp.

Gossett, F. R., 1950, rev. 1959: Manual of geodetic triangulation. *Special Publication* 247, U.S. Coast and Geodetic Survey, Washington, D.C. 363 pp. stock no. 003-002-00150-7).

Hoskinson, A. J. and Duerksen, J. A., 1952: Manual of geodetic astronomy. *Special Publication* 237, U.S. Coast and Geodetic Survey, Washington, D.C., 205 pp.

Meade, B. K., 1969: Corrections for refractive index as applied to electro-optical distance measurements. Environmental Science Services Administration (now NOAA), Rockville, Md., 27 pp.

Mitchell, H. C. and Simmons, L. G., 1945, rev. 1977: The State coordinate systems (a manual for surveyors). *Special Publication* 235, U.S. Coast and Geodetic Survey, Washington, D.C., 62 pp.

Mussetter, W., 1941, rev. 1959: Manual of reconnaissance for triangulation. *Special Publication* 225, U.S. Coast and Geodetic Survey, Washington, D.C., 105 pp.

Pfeifer, L., 1980: Input formats and specifications of the National Geodetic Survey data base, vol I: horizontal control data. *NOAA Manual* NOS NGS 2, National Oceanic and Atmospheric Administration, Rockville, Md. (in press). (Distribution is maintained by the National Geodetic Survey, NOS/NOAA, Rockville, MD 20852.)

Pfeifer, L. and Morrison, N., 1980: Input formats and specifications of the National Geodetic Survey data base, vol. II: vertical control data. *NOAA Manual* NOS NGS 2, National Oceanic and Atmospheric Administration, Rockville, Md. (in press). (Distribution is maintained by the National Geodetic Survey, NOS/NOAA, Rockville, MD 20852.)

Poling, A. C., 1947: Elevations from zenith distances (machine computation) with 6-place natural tangent tables, 0°–45°. *Publication* G-56, U.S. Coast and Geodetic Survey, Washington, D.C., 32 pp.

Rappleye, H. S., 1948a: Manual of leveling computation and adjustment. *Special Publication* 240, U.S. Coast and Geodetic Survey, Washington, D.C., 180 pp.

Rappleye, H. S., 1948b: Manual of geodetic leveling. *Special Publication* 239, U.S. Coast and Geodetic Survey, Washington, D.C. (Portions of text are obsolete.)

U.S. Army, 1970: Precise astronomic surveys. Department of the Army Technical Memorandum TM 5-442, 290 pp.

Whalen, C. T., 1978: Control leveling. *NOAA Technical Report* NOS 73 NGS 8, National Oceanic and Atmospheric Administration, Rockville, Md., 24 pp.

Whalen, C. T. and Balazs, E. I., 1976: Test results of first-order class III leveling. *NOAA Technical Report* NOS 68 NGS 4, National Oceanic and Atmospheric Administration, Rockville, Md., 30 pp.

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