Standards and Specifications for Geodetic Control Networks

Federal Geodetic Control Committee

Rear Adm. John D. Bossler, Chairman

Rockville, Maryland

September 1984

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Preface

This single publication is designed to replace both "Classification, Standards of Accuracy and General Specifications of Geodetic Control Surveys," issued February 1974, and "Specifications to Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys," issued June 1980. Because requirements and methods for acquisition of geodetic control are changing rapidly, this publication is being released in loose-leaf format so that it can be updated more conveniently and efficiently. Recipients of this publication wishing to receive updated information should complete and mail the form below. Comments on the contents and format of the publication are welcomed and should be addressed to:

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Contents

1. Introduction

The Government of the United States makes nationwide surveys, maps, and charts of various kinds. These are necessary to support the conduct of public business at all levels of government, for planning and carrying out national and local projects, the development and utilization of natural resources, national defense, land management, and monitoring crustal motion. Requirements for geodetic control surveys are most critical where intense development is taking place, particularly offshore areas, where surveys are used in the exploration and development of natural resources, and in delineation of state and international boundaries.

State and local governments and industry regularly cooperate in various parts of the total surveying and mapping program. In surveying and mapping large areas, it is first necessary to establish frameworks of horizontal, vertical, and gravity control. These provide a common basis for all surveying and mapping operations to ensure a coherent product. A reference system, or datum, is the set of numerical quantities that serves as a common basis. Three National Geodetic Control Networks have been created by the Government to provide the datums. It is the responsibility of the National Geodetic Survey (NGS) to actively maintain the National Geodetic Control Networks (appendix A).

These control networks consist of stable, identifiable points tied together by extremely accurate observations. From these observations, datum values (coordinates or gravity) are computed and published. These datum values provide the common basis that is so important to surveying and mapping activities.

As stated, the United States maintains three control networks. A horizontal network provides geodetic latitudes and longitudes in the North American Datum reference system; a vertical network furnishes elevations in the National Geodetic Vertical Datum reference system; and a gravity network supplies gravity values in the U.S. absolute gravity reference system. A given station may be a control point in one, two, or all three control networks.

It is not feasible for all points in the control networks to be of the highest possible accuracy. Different levels of accuracy are referred to as the "order" of a point. Orders are often subdivided further by a "class" designation. Datum values for a station are assigned an order (and class) based upon the appropriate classification standard for each of the three control networks. Horizontal and vertical standards are defined in reasonable conformance with past practice. The recent development of highly accurate absolute gravity instrumentation now allows a gravity reference standard. In the section on "Standards," the classification standards for each of the control networks are described, sample computations performed, and monumentation requirements given.

Control networks can be produced only by making very accurate measurements which are referred to identifiable control points. The combination of survey design, instrumentation, calibration procedures, observational techniques, and data reduction methods is known as a measurement system. The section on "Specifications" describes important components and states permissible tolerances for a variety of measurement systems.

Clearly, the control networks would be of little use if the datum values were not published. The section entitled "Information" describes the various products and formats of available geodetic data.

Upon request, the National Geodetic Survey will accept data submitted in the correct formats with the proper supporting documentation (appendix C) for incorporation into the national networks. When a survey is submitted for inclusion into the national networks, the survey measurements are processed in a quality control procedure that leads to their classification of accuracy and storage in the National Geodetic Survey data base. To fully explain the process we shall trace a survey from the planning stage to admission into the data base. This example will provide an overview of the standards and specifications, and how they work together.

The user should first compare the distribution and accuracy of current geodetic control with both immediate and long-term needs. From this basis, requirements for the extent and accuracy of the planned survey are determined. The classification standards of the control networks will help in this formulation. Hereafter, the requirements for the accuracy of the planned survey will be referred to as the "intended accuracy" of the survey. A measurement system is then chosen, based on various factors such as: distribution and accuracy of present control; region of the country; extent, distribution, and accuracy of the desired control; terrain and accessibility of control; and economic factors.

Upon selection of the measurement system, a survey design can be started. The design will be strongly dependent upon the "Network Geometry" specifications for that measurement system. Of particular importance is the requirement to connect to previously established control points. If this is not done, then the survey cannot be placed on the national datum. An adequate number of existing control point connections are often required in the specifications in order to ensure strong network geometry for other users of the control, and to provide several closure checks to help measure accuracy. NGS can certify the results of a survey only if it is connected to the national network.

Situations will arise where one cannot, or prefers not to, conform to the specifications. NGS may downgrade the classification of a survey based upon failure to adhere to the measurement system specifications if the departure degrades the precision, accuracy, or utility of the survey. On the other hand, if specification requirements for the desired level of accuracy are exceeded, it may be possible to upgrade a survey to a higher classification.

Depending upon circumstances, one may wish to go into the field to recover old control and perform reconnaissance and site inspection for the new survey. Monumentation may be performed at this stage. Instruments should be checked to conform to the "Instrumentation" specifications, and to meet the "Calibration Procedures" specifications. Frequent calibration is an excellent method to help ensure accurate surveys.

In the field, the "Field Procedures" specifications are used to guide the methods for taking survey measurements. It must be stressed that the "Field Procedures" section is not an exhaustive account of how to perform observations. Reference should be made also to the appropriate manuals of observation methods and instruments.

Computational checks can be found in the "Field Procedures" as well as in the "Office Procedures" specifications, since one will probably want to perform some of the computations in the field to detect blunders. It is not necessary for the user to do the computations described in the "Office Procedures" specifications, since they will be done by NGS. However, it is certainly in the interest of the user to compute these checks before leaving the field, in case reobservations are necessary. With the tremendous increase in programmable calculator and small computer technology, any of the computations in the "Office Procedures" specifications could be done with ease in the field.

At this point the survey measurements have been collected, together with the new description and recovery notes of the stations in the new survey. They are then placed into the formats specified in the Federal Geodetic Control Committee (FGCC) publications *Input Formats and Specifications of the National Geodetic Survey Data Base*. Further details of this process can be found in appendix C, "Procedures for Submitting Data to the National Geodetic Survey."

The data and supporting documentation, after being received at NGS, are processed through a quality control procedure to make sure that all users may place confidence in the new survey points. First, the data and documentation are examined for compliance with the measurement system specifications for the intended accuracy of the new survey. Then office computations are performed, including a minimally constrained least squares adjustment. (See appendix B for details.) From this adjustment, accuracy measures can be computed by error propagation. The accuracy classification thus computed is called the "provisional accuracy" of the survey.

The provisional accuracy is compared to the intended accuracy. The difference indicates the departure of the accuracy of the survey from the specifications. If the difference is small, the intended accuracy has precedence because a possible shift in classification is not warranted. However, if the difference is substantial, the provisional accuracy will supersede the intended accuracy, either as a downgrade or an upgrade.

As the final step in the quality control procedure, the variance factor ratio computation using established control, as explained in the section on "Standards," is determined for the new survey. If this result meets the criteria stated there, then the survey is classified in accordance with the provisional accuracy (or intended accuracy, whichever has precedence).

Cases arise where the variance factor ratio is significantly larger than expected. Then the control network is at fault, or the new survey is subject to some unmodeled error source which degrades its accuracy. Both the established control measurements and the new survey measurements will be scrutinized by NGS to determine the source of the problem. In difficult cases, NGS may make diagnostic measurements in the field.

Upon completion of the quality control check, the survey measurements and datum values are placed into the data base. They become immediately available for electronic retrieval, and will be distributed in the next publication cycle by the National Geodetic Information Branch of NGS.

A final remark bears on the relationship between the classification standards and measurement system specifications. Specifications are combinations of rules of thumb and studies of error propagation, based upon experience, of how to best achieve a desired level of quality. Unfortunately, there is no guarantee that a particular standard will be met if the associated specifications are followed. However, the situation is ameliorated by a safety factor of two incorporated in the standards and specifications. Because of this safety factor, it is possible that one may fail to meet the specifications and still satisfy the desired standard. This is why the geodetic control is not automatically downgraded when one does not adhere to the specifications. Slight departures from the specifications can be accommodated. In practice, one should always strive to meet the measurement system specifications when extending a National Geodetic Control Network.

2. Standards

The classification standards of the National Geodetic Control Networks are based on accuracy. This means that when control points in a particular survey are classified, they are certified as having datum values consistent with all other points in the network, not merely those within that particular survey. It is not observation closures within a survey which are used to classify control points, but the ability of that survey to duplicate already established control values. This comparison takes into account models of crustal motion, refraction, and any other systematic effects known to influence the survey measurements.

The NGS procedure leading to classification covers four steps:

1. The survey measurements, field records, sketches, and other documentation are examined to verify compliance with the specifications for the intended accuracy of the survey. This examination may lead to a modification of the intended accuracy.
2. Results of a minimally constrained least squares adjustment of the survey measurements are examined to ensure correct weighting of the observations and freedom from blunders.
3. Accuracy measures computed by random error propagation determine the provisional accuracy. If the provisional accuracy is substantially different from the intended accuracy of the survey, then the provisional accuracy supersedes the intended accuracy.
4. A variance factor ratio for the new survey combined with network data is computed by the Iterated Almost Unbiased Estimator (IAUE) method (appendix B). If the variance factor ratio is reasonably close to 1.0 (typically less than 1.5), then the survey is considered to check with the network, and the survey is classified with the provisional (or intended) accuracy. If the variance factor ratio is much greater than 1.0 (typically 1.5 or greater), then the survey is considered to not check with the network, and both the survey and network measurements will be scrutinized for the source of the problem.

2.1 Horizontal Control Network Standards

When a horizontal control point is classified with a particular order and class, NGS certifies that the geodetic latitude and longitude of that control point bear a relation of specific accuracy to the coordinates of all other points in the horizontal control network. This relation is expressed as a distance accuracy, 1:a. A distance accuracy is the ratio of the relative positional error of a pair of control points to the horizontal separation of those points.

Table 2.1—Distance accuracy standards

Classification	*Minimum distance accuracy*
First-order	1:100,000
Second-order, class I	1: 50,000
Second-order, class II	1: 20,000
Third-order, class I	1: 10,000
Third-order, class II	1: 5,000

A distance accuracy, 1:a, is computed from a minimally constrained, correctly weighted, least squares adjustment by:

$$a = d/s$$

where
a = distance accuracy denominator
s = propagated standard deviation of distance between survey points obtained from the least squares adjustment
d = distance between survey points

The distance accuracy pertains to all pairs of points (but in practice is computed for a sampling of pairs of points). The worst distance accuracy (smallest denominator) is taken as the provisional accuracy. If this is substantially larger or smaller than the intended accuracy, then the provisional accuracy takes precedence.

As a test for systematic errors, the variance factor ratio of the new survey is computed by the Iterated Almost Unbiased Estimator (IAUE) method described in appendix B. This computation combines the new survey measurements with existing network data, which are assumed to be correctly weighted and free of systematic error. If the variance factor ratio is substantially greater than unity then the survey does not check with the network, and both the survey and the network data will be examined by NGS.

Computer simulations performed by NGS have shown that a variance factor ratio greater than 1.5 typically indicates systematic errors between the survey and the network. Setting a cutoff value higher than this could allow undetected systematic error to propagate into the national network. On the other hand, a higher cutoff value might be considered if the survey has only a small number of connections to the network, because this circumstance would tend to increase the variance factor ratio.

In some situations, a survey has been designed in which different sections provide different orders of control. For these multi-order surveys, the computed distance accuracy denominators should be grouped into sets appropriate to the different parts of the survey. Then, the smallest value of a in each set is used to classify the control points of that portion, as discussed above. If there are sufficient connections to the network, several variance factor ratios, one for each section of the survey, should be computed.

Horizontal Example

Suppose a survey with an intended accuracy of first-order (1:100,000) has been performed. A series of propagated distance accuracies from a minimally constrained adjustment is now computed.

Line	*s* *(m)*	*d* *(m)*	*1:a*
1-2	0.141	17,107	1:121,326
1-3	0.170	20,123	1:118,371
2-3	0.164	15,505	1: 94,543
...	.	.	.
...	.	.	.
...	.	.	.

Suppose that the worst distance accuracy is 1:94,543. This is not substantially different from the intended accuracy of 1:100,000, which would therefore have precedence for classification. It is not feasible to precisely quantify "substantially different." Judgment and experience are determining factors.

Now assume that a solution combining survey and network data has been obtained (as per appendix B), and that a variance factor ratio of 1.2 was computed for the survey. This would be reasonably close to unity, and would indicate that the survey checks with the network. The survey would then be classified as first-order using the intended accuracy of 1:100,000.

However, if a variance factor of, say, 1.9 was computed, the survey would not check with the network. Both the survey and network measurements then would have to be scrutinized to find the problem.

Monumentation

Control points should be part of the National Geodetic Horizontal Network only if they possess permanence, horizontal stability with respect to the Earth's crust, and a horizontal location which can be defined as a point. A 30-centimeter-long wooden stake driven into the ground, for example, would lack both permanence and horizontal stability. A mountain peak is difficult to define as a point. Typically, corrosion resistant metal disks set in a large concrete mass have the necessary qualities. First-order and second-order, class I, control points should have an underground mark, at least two monumented reference marks at right angles to one another, and at least one monumented azimuth mark no less than 400 m from the control point. Replacement of a temporary mark by a more permanent mark is not acceptable unless the two marks are connected in timely fashion by survey observations of sufficient accuracy. Detailed information may be found in C&GS *Special Publication* 247, "Manual of geodetic triangulation."

2.2 Vertical Control Network Standards

When a vertical control point is classified with a particular order and class, NGS certifies that the orthometric elevation at that point bears a relation of specific accuracy to the elevations of all other points in the vertical control network. That relation is expressed as an elevation difference accuracy, b. An elevation difference accuracy is the relative elevation error between a pair of control points that is scaled by the square root of their horizontal separation traced along existing level routes.

Table 2.2—Elevation accuracy standards

Classification	*Maximum elevation difference accuracy*
First-order, class I	0.5
First-order, class II	0.7
Second-order, class I	1.0
Second-order, class II	1.3
Third-order	2.0

An elevation difference accuracy, b, is computed from a minimally constrained, correctly weighted, least squares adjustment by

$$b = S/\sqrt{d}$$

where

d = approximate horizontal distance in kilometers between control point positions traced along existing level routes.

S = propagated standard deviation of elevation difference in millimeters between survey control points obtained from the least squares adjustment. Note that the units of b are (mm)/$\sqrt{}$(km).

The elevation difference accuracy pertains to all pairs of points (but in practice is computed for a sample). The worst elevation difference accuracy (largest value) is taken as the provisional accuracy. If this is substantially larger or smaller than the intended accuracy, then the provisional accuracy takes precedence.

As a test for systematic errors, the variance factor ratio of the new survey is computed by the Iterated Almost Unbiased Estimator (IAUE) method described in appendix B. This computation combines the new survey measurements with existing network data, which are assumed to be correctly weighted and free of systematic error. If the variance factor ratio is substantially greater than unity, then the survey does not check with the network, and both the survey and the network data will be examined by NGS.

Computer simulations performed by NGS have shown that a variance factor ratio greater than 1.5 typically indicates systematic errors between the survey and the network. Setting a cutoff value higher than this could allow undetected systematic error to propagate into the national network. On the other hand, a higher cutoff value might be considered if the survey has only a small number of connections to the network, because this circumstance would tend to increase the variance factor ratio.

In some situations, a survey has been designed in which different sections provide different orders of control. For these multi-order surveys, the computed elevation difference accuracies should be grouped into sets appropriate to the different parts of the survey. Then, the largest value of b in each set is used to classify the control points of that portion, as discussed above. If there are sufficient connections to the network, several variance factor ratios, one for each section of the survey, should be computed.

Vertical Example

Suppose a survey with an intended accuracy of second-order, class II has been performed. A series of propagated elevation difference accuracies from a minimally constrained adjustment is now computed.

Line	*S (mm)*	*d (km)*	*b (mm)/√(km)*
1-2	1.574	1.718	1.20
1-3	1.743	2.321	1.14
2-3	2.647	4.039	1.32
...	.	.	.
...	.	.	.
...	.	.	.

Suppose that the worst elevation difference accuracy is 1.32. This is not substantially different from the intended accuracy of 1.3 which would therefore have precedence for classification. It is not feasible to precisely quantify "substantially different." Judgment and experience are determining factors.

Now assume that a solution combining survey and network data has been obtained (as per appendix B), and that a variance factor ratio of 1.2 was computed for the survey. This would be reasonably close to unity and would indicate that the survey checks with the network. The survey would then be classified as second-order, class II, using the intended accuracy of 1.3.

However, if a survey variance factor ratio of, say, 1.9 was computed, the survey would not check with the network. Both the survey and network measurements then would have to be scrutinized to find the problem.

Monumentation

Control points should be part of the National Geodetic Vertical Network only if they possess permanence, vertical stability with respect to the Earth's crust, and a vertical location that can be defined as a point. A 30-centimeter-long wooden stake driven into the ground, for example, would lack both permanence and vertical stability. A rooftop lacks stability and is difficult to define as a point. Typically, corrosion resistant metal disks set in large rock outcrops or long metal rods driven deep into the ground have the necessary qualities. Replacement of a temporary mark by a more permanent mark is not acceptable unless the two marks are connected in timely fashion by survey observations of sufficient accuracy. Detailed information may be found in *NOAA Manual NOS NGS* 1, "Geodetic bench marks."

2.3 Gravity Control Network Standards

When a gravity control point is classified with a particular order and class, NGS certifies that the gravity value at that control point possesses a specific accuracy.

Gravity is commonly expressed in units of milligals (mGal) or microgals (μGal) equal, respectively, to (10^{-5}) meters/sec^2, and (10^{-8}) meters/sec^2. Classification order refers to measurement accuracies and class to site stability.

Table 2.3—Gravity accuracy standards

Classification	*Gravity accuracy (μGal)*
First-order, class I	20 (subject to stability verification)
First-order, class II	20
Second-order	50
Third-order	100

When a survey establishes only new points, and where only absolute measurements are observed, then each survey point is classified independently. The standard deviation from the mean of measurements observed at that point is corrected by the error budget for noise sources in accordance with the following formula:

$$c^2 = \sum_{i+1}^{n} \frac{(x_i - x_m)^2}{n-1} + e^2$$

where

c =gravity accuracy

x_i=gravity measurement

n =number of measurements

$$x_m = (\sum_{i=1}^{n} x_i)/n$$

e=external random error

The value obtained for c is then compared directly against the gravity accuracy standards table.

When a survey establishes points at which both absolute and relative measurements are made, the absolute determination ordinarily takes precedence and the point is classified accordingly. (However, see Example D below for an exception.)

When a survey establishes points where only relative measurements are observed, and where the survey is tied to the National Geodetic Gravity Network, then the gravity accuracy is identified with the propagated gravity standard deviation from a minimally constrained, correctly weighted, least squares adjustment.

The worst gravity accuracy of all the points in the survey is taken as the provisional accuracy. If the provisional accuracy exceeds the gravity accuracy limit set for the intended survey classification, then the survey is classified using the provisional accuracy.

As a test for systematic errors, the variance factor ratio of the new survey is computed by the Iterated Almost Unbiased Estimator (IAUE) method described in appendix B. This computation combines the new survey measurements with existing network data which are assumed to be correctly weighted and free of systematic error. If the variance factor ratio is substantially greater than unity, then the survey does not check with the network, and both the survey and the network data will be examined by NGS.

Computer simulations performed by NGS have shown that a variance factor ratio greater than 1.5 typically indicates systematic errors between the survey and the network. Setting a cutoff value higher than this could allow undetected systematic error to propagate into the national network. On the other hand, a higher cutoff value might be considered if the survey has only a minimal number of connections to the network, because this circumstance would tend to increase the variance factor ratio.

In some situations, a survey has been designed in which different sections provide different orders of control. For these multi-order surveys, the computed gravity accuracies should be grouped into sets appropriate to the different parts of the survey. Then, the largest value of c in each set is used to classify the control points of that portion, as discussed above. If there are sufficient connections to the network, several variance factor ratios, one for each part of the survey, should be computed.

Gravity Examples

Example A. Suppose a gravity survey using absolute measurement techniques has been performed. These points are then unrelated. Consider one of these survey points.

Assume n = 750

$$\sum_{i=1}^{750} (x_i - x_m)^2 = .169 \text{ mGal}^2$$

$$e = 5\ \mu\text{Gal}$$

$$c^2 = \frac{0.169}{750-1} + (.005)^2$$

$$c = 16\ \mu\text{Gal}$$

The point is then classified as first-order, class II.

Example B. Suppose a relative gravity survey with an intended accuracy of second-order (50 μGal) has been performed. A series of propagated gravity accuracies from a minimally constrained adjustment is now computed.

Station	*Gravity standard deviation (μGal)*
1	38
2	44
3	55
.	.
.	.
.	.

Suppose that the worst gravity accuracy was 55 μGal. This is worse than the intended accuracy of 50 μGal. Therefore, the provisional accuracy of 55 μGal would have precedence for classification, which would be set to third-order.

Now assume that a solution combining survey and network data has been obtained (as per appendix B) and that a variance factor of 1.2 was computed for the survey. This would be reasonably close to unity, and would indicate that the survey checks with the network. The survey would then be classified as third-order using the provisional accuracy of 55 μGal.

However, if a variance factor of, say, 1.9 was computed, the survey would not check with the network. Both the survey and network measurements then would have to be scrutinized to find the problem.

Example C. Suppose a survey consisting of both absolute and relative measurements has been made at the same points. Assume the absolute observation at one of the points yielded a classification of first-order, class II, whereas the relative measurements produced a value to second-order standards. The point in question would be classified as first-order, class II, in accordance with the absolute observation.

Example D. Suppose we have a survey similar to Case C, where the absolute measurements at a particular point yielded a third-order classification due to an unusually noisy observation session, but the relative measurements still satisfied the second-order standard. The point in question would be classified as second-order, in accordance with the relative measurements.

Monumentation

Control points should be part of the National Geodetic Gravity Network only if they possess permanence, horizontal and vertical stability with respect to the Earth's crust, and a horizontal and vertical location which can be defined as a point. For all orders of accuracy, the mark should be imbedded in a stable platform such as flat, horizontal concrete. For first-order, class I stations, the platform should be imbedded in stable, hard rock, and checked at least twice for the first year to ensure stability. For first-order, class II stations, the platform should be located in an extremely stable environment, such as the concrete floor of a mature structure. For second and third-order stations, standard bench mark monumentation is adequate. Replacement of a temporary mark by a more permanent mark is not acceptable unless the two marks are connected in timely fashion by survey observations of sufficient accuracy. Detailed information is given in *NOAA Manual NOS NGS* 1, "Geodetic bench marks." Monuments should not be near sources of electromagnetic interference.

It is recommended, but not necessary, to monument third-order stations. However, the location associated with the gravity value should be recoverable, based upon the station description.

3. Specifications

3.1 Introduction

All measurement systems regardless of their nature have certain common qualities. Because of this, the measurement system specifications follow a prescribed structure as outlined below. These specifications describe the important components and state permissible tolerances used in a general context of accurate surveying methods. The user is cautioned that these specifications are not substitutes for manuals that detail recommended field operations and procedures.

The observations will have spatial or temporal relationships with one another as given in the "Network Geometry" section. In addition, this section specifies the frequency of incorporation of old control into the survey. Computer simulations could be performed instead of following the "Network Geometry" and "Field Procedures" specifications. However, the user should consult the National Geodetic Survey before undertaking such a departure from the specifications.

The "Instrumentation" section describes the types and characteristics of the instruments used to make observations. An instrument must be able to attain the precision requirements given in "Field Procedures."

The section "Calibration Procedures" specifies the nature and frequency of instrument calibration. An instrument must be calibrated whenever it has been damaged or repaired.

The "Field Procedures" section specifies particular rules and limits to be met while following an appropriate method of observation. For a detailed account of how to perform observations, the user should consult the appropriate manuals.

Since NGS will perform the computations described under "Office Procedures," it is not necessary for the user to do them. However, these computations provide valuable checks on the survey measurements that could indicate the need for some reobservations. This section specifies commonly applied corrections to observations, and computations which monitor the precision and accuracy of the survey. It also discusses the correctly weighted, minimally constrained least squares adjustment used to ensure that the survey work is free from blunders and able to achieve the intended accuracy. Results of the least squares adjustment are used in the quality control and accuracy classification procedures. The adjustment performed by NGS will use models of error sources, such as crustal motion, when they are judged to be significant to the level of accuracy of the survey.

3.2 Triangulation

Triangulation is a measurement system comprised of joined or overlapping triangles of angular observations supported by occasional distance and astronomic observations. Triangulation is used to extend horizontal control.

Network Geometry

Order *Class*	*First*	*Second* *I*	*Second* *II*	*Third* *I*	*Third* *II*
Station spacing not less than (km)	15	10	5	0.5	0.5
Average minimum distance angle† of figures not less than	40°	35°	30°	30°	25°
Minimum distance angle† of all figures not less than	30°	25°	25°	20°	20°
Base line spacing not more than (triangles)	5	10	12	15	15
Astronomic azimuth spacing not more than (triangles)	8	10	10	12	15

† Distance angle is angle opposite the side through which distance is propagated.

The new survey is required to tie to at least four network control points spaced well apart. These network points must have datum values equivalent to or better than the intended order (and class) of the new survey. For example, in an arc of triangulation, at least two network control points should be occupied at each end of the arc. Whenever the distance between two new unconnected survey points is less than 20 percent of the distance between those points traced along existing or new connections, then a direct connection should be made between those two survey points. In addition, the survey should tie into any sufficiently accurate network control points within the station spacing distance of the survey. These network stations should be occupied and sufficient observations taken to make these stations integral parts of the survey. Nonredundant geodetic connections to the network stations are not considered sufficient ties. Nonredundantly determined stations are not allowed. Control stations should not be determined by intersection or resection methods. Simultaneous reciprocal vertical angles or geodetic leveling are observed along base lines. A base line need not be observed if other base lines of sufficient accuracy were observed within the base line spacing specification in the network, and similarly for astronomic azimuths.

Instrumentation

Only properly maintained theodolites are adequate for observing directions and azimuths for triangulation. Only precisely marked targets, mounted stably on tripods or supported towers, should be employed. The target should have a clearly defined center, resolvable at the minimum control spacing. Optical plummets or collimators are required to ensure that the theodolites and targets are centered over the marks. Microwave-type electronic distance measurement (EDM) equipment is not sufficiently accurate for measuring higher-order base lines.

Order Class	*First*	*Second I*	*Second II*	*Third I*	*Third II*
Theodolite, least count	0.2″	0.2″	1.0″	1.0″	1.0″

Calibration Procedures

Each year and whenever the difference between direct and reverse readings of the theodolite depart from 180° by more than 30″, the instrument should be adjusted for collimation error. Readjustment of the cross hairs and the level bubble should be done whenever their misadjustments affect the instrument reading by the amount of the least count.

All EDM devices and retroreflectors should be serviced regularly and checked frequently over lines of known distances. The National Geodetic Survey has established specific calibration base lines for this purpose. EDM instruments should be calibrated annually, and frequency checks made semiannually.

Field Procedures

Theodolite observations for first-order and second-order, class I surveys may only be made at night. Reciprocal vertical angles should be observed at times of best atmospheric conditions (between noon and late afternoon) for all orders of accuracy. Electronic distance measurements need a record at both ends of the line of wet and dry bulb temperatures to ±1°C, and barometric pressure to ±5 mm of mercury. The theodolite and targets should be centered to within 1 mm over the survey mark or eccentric point.

Order Class	*First*	*Second I*	*Second II*	*Third I*	*Third II*
Directions					
Number of positions	16	16	8 or 12†	4	2
Standard deviation of mean not to exceed	0.4″	0.5″	0.8″	1.2″	2.0″
Rejection limit from the mean	4″	4″	5″	5″	5″
Reciprocal Vertical Angles (along distance sight path)					
Number of independent observations direct/reverse	3	3	2	2	2
Maximum spread	10″	10″	10″	10″	20″
Maximum time interval between reciprocal angles (hr)	1	1	1	1	1
Astronomic Azimuths					
Observations per night	16	16	16	8	4
Number of nights	2	2	1	1	1
Standard deviation of mean not to exceed	0.45″	0.45″	0.6″	1.0″	1.7″
Rejection limit from the mean	5″	5″	5″	6″	6″
Electro-Optical Distances					
Minimum number of days	2*	2*	1	1	1
Minimum number of measurements/day	2§	2§	2§	1	1
Minimum number of concentric observations/measurement	2	2	1	1	1
Minimum number of offset observations/measurement	2	2	2	1	1
Maximum difference from mean of observations (mm)	40	40	50	60	60
Minimum number of readings/observation (or equivalent)	10	10	10	10	10
Maximum difference from mean of readings (mm)	‡	‡	‡	‡	‡
Infrared Distances					
Minimum number of days	—	2*	1	1	1
Minimum number of measurements	—	2§	2§	1	1
Minimum number of concentric observations/measurement	—	1	1	1	1
Minimum number of offset observations/measurement	—	2	1	1	1
Maximum difference from mean of observations (mm)	—	5	5	10	10
Minimum number of readings/observation (or equivalent)	—	10	10	10	10
Maximum difference from mean of readings (mm)	—	‡	‡	‡	‡
Microwave Distances					
Minimum number of measurements	—	—	—	2	1
Minimum time span between measurements (hr)	—	—	—	8	—

Order Class	*First*	*Second I*	*Second II*	*Third I*	*Third II*
Maximum difference between measurements (mm)	—	—	—	100	—
Minimum number of concentric observations/measurement	—	—	—	2**	1**
Maximum difference from mean of observations (mm)	—	—	—	100	150
Minimum number of readings/observation (or equivalent)	—	—	—	20	20
Maximum difference from mean of readings (mm) ..	—	—	—	‡	‡

† 8 if 0.2", 12 if 1.0" resolution.
* two or more instruments.
§ one measurement at each end of the line.
‡ as specified by manufacturer.
** carried out at both ends of the line.

Measurements of astronomic latitude and longitude are not required in the United States, except perhaps for first-order work, because sufficient information for determining deflections of the vertical exists. Detailed procedures can be found in Hoskinson and Duerksen (1952).

Office Procedures

Order Class	*First*	*Second I*	*Second II*	*Third I*	*Third II*
Triangle Closure					
Average not to exceed	1.0"	1.2"	2.0"	3.0"	5.0"
Maximum not to exceed	3"	3"	5"	5"	10"
Side Checks					
Mean absolute correction by side equation not to exceed	0.3"	0.4"	0.6"	0.8"	2.0"

A minimally constrained least squares adjustment will be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Distance standard errors computed by error propagation in this correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models which account for the following:

semimajor axis of the ellipsoid ....($a = 6378137$ m)
reciprocal flattening of the ellipsoid ....($1/f = 298.257222$)
mark elevation above mean sea level........................(known to ±1 m)
geoid heights ....(known to ±6 m)
deflections of the vertical ....(known to ±3")
geodesic correction
skew normal correction
height of instrument
height of target
sea level correction
arc correction
geoid height correction
second velocity correction
crustal motion

3.3 Traverse

Traverse is a measurement system comprised of joined distance and theodolite observations supported by occasional astronomic observations. Traverse is used to densify horizontal control.

Network Geometry

Order Class	*First*	*Second I*	*Second II*	*Third I*	*Third II*
Station spacing not less than (km)	10	4	2	0.5	0.5
Maximum deviation of main traverse from straight line	20°	20°	25°	30°	40°
Minimum number of bench mark ties	2	2	2	2	2
Bench mark tie spacing not more than (segments)	6	8	10	15	20
Astronomic azimuth spacing not more than (segments)	6	12	20	25	40
Minimum number of network control points	4	3	2	2	2

The new survey is required to tie to a minimum number of network control points spaced well apart. These network points must have datum values equivalent to or better than the intended order (and class) of the new survey. Whenever the distance between two new unconnected survey points is less than 20 percent of the distance between those points traced along existing or new connections, then a direct connection must be made between those two survey points. In addition, the survey should tie into any sufficiently accurate network control points within the station spacing distance of the survey. These ties must include EDM or taped distances. Nonredundant geodetic connections to the network stations are not considered sufficient ties. Nonredundantly determined stations are not allowed. Reciprocal vertical angles or geodetic leveling are observed along all traverse lines.

Instrumentation

Only properly maintained theodolites are adequate for observing directions and azimuths for traverse. Only precisely marked targets, mounted stably on tripods or supported towers, should be employed. The target should have a clearly defined center, resolvable at the minimum control spacing. Optical plummets or collimators are required to ensure that the theodolites and targets are centered over the marks. Microwave-type electronic distance measurement equipment is not sufficiently accurate for measuring first-order traverses.

Order Class	First	Second I	Second II	Third I	Third II
Theodolite, least count	0.2"	1.0"	1.0"	1.0"	1.0"

Calibration Procedures

Each year and whenever the difference between direct and reverse readings of the theodolite depart from 180° by more than 30", the instrument should be adjusted for collimation error. Readjustment of the cross hairs and the level bubble should be done whenever their misadjustments affect the instrument reading by the amount of the least count.

All electronic distance measuring devices and retroreflectors should be serviced regularly and checked frequently over lines of known distances. The National Geodetic Survey has established specific calibration base lines for this purpose. EDM instruments should be calibrated annually, and frequency checks made semiannually.

Field Procedures

Theodolite observations for first-order and second-order, class I surveys may be made only at night. Electronic distance measurements need a record at both ends of the line of wet and dry bulb temperatures to ±1°C and barometric pressure to ±5 mm of mercury. The theodolite, EDM, and targets should be centered to within 1 mm over the survey mark or eccentric point.

Order Class	First	Second I	Second II	Third I	Third II
Directions					
Number of positions	16	8 or 12†	6 or 8*	4	2
Standard deviation of mean not to exceed	0.4"	0.5"	0.8"	1.2"	2.0"
Rejection limit from the mean	4"	5"	5"	5"	5"
Reciprocal Vertical Angles (along distance sight path)					
Number of independent observations direct/reverse	3	3	2	2	2
Maximum spread	10"	10"	10"	10"	20"
Maximum time interval between reciprocal angles (hr)	1	1	1	1	1
Astronomic Azimuths					
Observations per night	16	16	12	8	4
Number of nights	2	2	1	1	1
Standard deviation of mean not to exceed	0.45"	0.45"	0.6"	1.0"	1.7"
Rejection limit from the mean	5"	5"	5"	6"	6"
Electro-Optical Distances					
Minimum number of measurements	1	1	1	1	1
Minimum number of concentric observations/measurement	1	1	1	1	1
Minimum number of offset observations/measurement	1	1	—	—	—
Maximum difference from mean of observations (mm)	60	60	—	—	—
Minimum number of readings/observation (or equivalent)	10	10	10	10	10
Maximum difference from mean of readings (mm)	§	§	§	§	§
Infrared Distances					
Minimum number of measurements	1	1	1	1	1
Minimum number of concentric observations/measurement	1	1	1	1	1
Minimum number of offset observations/measurement	1	1	1‡	—	—
Maximum difference from mean of observations (mm)	10	10	10‡	—	—
Minimum number of readings/observation	10	10	10	10	10
Maximum difference from mean of readings (mm)	§	§	§	§	§
Microwave Distances					
Minimum number of measurements	—	1	1	1	1
Minimum number of concentric observations/measurement	—	2**	1**	1**	1**
Maximum difference from mean of observations (mm)	—	150	150	200	200
Minimum number of readings/observation	—	20	20	10	10
Maximum difference from mean of readings (mm)	—	§	§	§	§

† 8 if 0.2", 12 if 1.0" resolution.
* 6 if 0.2", 8 if 1.0" resolution.
§ as specified by manufacturer.
‡ only if decimal reading near 0 or high 9's.
** carried out at both ends of the line.

Measurements of astronomic latitude and longitude are not required in the United States, except perhaps for first-order work, because sufficient information for determining deflections of the vertical exists. Detailed procedures can be found in Hoskinson and Duerksen (1952).

Office Procedures

Order Class	First	Second I	Second II	Third I	Third II
Azimuth closure at azimuth check point (seconds of arc)	$1.7\sqrt{N}$	$3.0\sqrt{N}$	$4.5\sqrt{N}$	$10.0\sqrt{N}$	$12.0\sqrt{N}$
Position closure after azimuth adjustment†	$0.04\sqrt{K}$ or 1:100,000	$0.08\sqrt{K}$ or 1:50,000	$0.20\sqrt{K}$ or 1:20,000	$0.40\sqrt{K}$ or 1:10,000	$0.80\sqrt{K}$ or 1:5,000

(N is number of segments, K is route distance in km)
† The expression containing the square root is designed for longer lines where higher proportional accuracy is required. Use the formula that gives the smallest permissible closure. The closure (e.g., 1:100,000) is obtained by computing the difference between the computed and fixed values, and dividing this difference by K.
Note: Do not confuse closure with distance accuracy of the survey.

A minimally constrained least squares adjustment will be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Distance standard errors computed by error propagation in a correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models which account for the following:

semimajor axis of the ellipsoid	($a = 6378137$ m)
reciprocal flattening of the ellipsoid	($1/f = 298.257222$)
mark elevation above mean sea level	(known to ± 1 m)
geoid heights	(known to ± 6 m)
deflections of the vertical	(known to $\pm 3''$)
geodesic correction	
skew normal correction	
height of instrument	
height of target	
sea level correction	
arc correction	
geoid height correction	
second velocity correction	
crustal motion	

3.4 Inertial Surveying

Inertial surveying is a measurement system comprised of lines, or a grid, of Inertial Surveying System (ISS) observations. These specifications cover use of inertial systems only for horizontal control.

Network Geometry

Order *Class*	*Second* *I*	*Second* *II*	*Third* *I*	*Third* *II*
Station spacing not less than (km)	10	4	2	1
Maximum deviation from straight line connecting endpoints	20°	25°	30°	35°

Each inertial survey line is required to tie into a minimum of four horizontal network control points spaced well apart and should begin and end at network control points. These network control points must have horizontal datum values better than the intended order (and class) of the new survey. Whenever the shortest distance between two new unconnected survey points is less than 20 percent of the distance between those points traced along existing or new connections, then a direct connection should be made between those two survey points. In addition, the survey should connect to any sufficiently accurate network control points within the distance specified by the station spacing. The connections may be measured by EDM or tape traverse, or by another ISS line. If an ISS line is used, then these lines should follow the same specifications as all other ISS lines in the survey.

For extended area surveys by ISS, a grid of intersecting lines that satisfies the 20 percent rule stated above can be designed. There must be a mark at each intersection of the lines. This mark need not be a permanent monument; it may be a stake driven into the ground. For a position to receive an accuracy classification, it must be permanently monumented.

A grid of intersecting lines should contain a minimum of eight network points, and should have a network control point at each corner. The remaining network control points may be distributed about the interior or the periphery of the grid. However, there should be at least one network control point at an intersection of the grid lines near the center of the grid. If the required network points are not available, then they should be established by some other measurement system. Again, the horizontal datum values of these network control points must have an order (and class) better than the intended order (and class) of the new survey.

Instrumentation

ISS equipment falls into two types: analytic (or strapdown) and semianalytic. Analytic inertial units are not considered to possess geodetic accuracy. Semianalytic units are either "space stable" or "local level." Space stable systems maintain the orientation of the platform with respect to inertial space. Local level systems continuously torque the accelerometers to account for Earth rotation and movement of the inertial unit, and also torque the platform to coincide with the local level. This may be done on command at a coordinate update, or whenever the unit achieves zero velocity (Zero velocity UPdaTe, or "ZUPT"). Independently of the measurement technique, the recorded data may be filtered by an onboard computer. Because of the variable quality of individual ISS instruments, the user should test an instrument with existing geodetic control beforehand.

An offset measurement device accurate to within 5 mm should be affixed to the inertial unit or the vehicle.

Calibration Procedures

A static calibration should be performed yearly and immediately after repairs affecting the platform, gyroscopes, or accelerometers.

A dynamic or field calibration should be performed prior to each project or subsequent to a static calibration. The dynamic calibration should be performed only between horizontal control points of first-order accuracy and in each cardinal direction. The accelerometer scale factors from this calibration should be recorded and, if possible, stored in the onboard computer of the inertial unit.

Before each project or after repairs affecting the offset measurement device or the inertial unit, the relation between the center of the inertial unit and the zero point of the offset measurement device should be established.

Field Procedures

When surveying in a helicopter, the helicopter must come to rest on the ground for all ZUPT's and all measurements.

Order Class	Second I	Second II	Third I	Third II
Minimum number of complete runs per line	2	1	1	1
Maximum deviation from a uniform rate of travel (including ZUPT)	15%	20%	25%	30%
Maximum ZUPT interval (ZUPT to ZUPT) (sec)	200	240	300	300

A complete ISS measurement consists of measurement of the line while traveling in one direction, followed by measurement of the same line while traveling in the reverse direction (double-run). A coordinate update should not be performed at the far point or at midpoints of a line, even though those coordinates may be known.

The mark offset should be measured to the nearest 5 mm.

Office Procedures

Order Class	Second I	Second II	Third I	Third II
Maximum difference of smoothed coordinates between forward and reverse run (cm)	60	60	70	80

A minimally constrained least squares adjustment of the raw or filtered survey data will be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Distance standard errors computed by error propagation in this correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use the best available model for the particular inertial system. Weighted averages of individually smoothed lines are not considered substitutes for a combined least squares adjustment to achieve geodetic accuracy.

3.5 Geodetic Leveling

Geodetic leveling is a measurement system comprised of elevation differences observed between nearby rods. Leveling is used to extend vertical control.

Network Geometry

Order Class	First I	First II	Second I	Second II	Third
Bench mark spacing not more than (km)	3	3	3	3	3
Average bench mark spacing not more than (km)	1.6	1.6	1.6	3.0	3.0

Order Class	First I	First II	Second I	Second II	Third
Line length between network control points not more than (km)	300	100	50	50 (double-run) 25 (single-run)	25 10

New surveys are required to tie to existing network bench marks at the beginning and end of the leveling line. These network bench marks must have an order (and class) equivalent to or better than the intended order (and class) of the new survey. First-order surveys are required to perform check connections to a minimum of six bench marks, three at each end. All other surveys require a minimum of four check connections, two at each end. "Check connection" means that the observed elevation difference agrees with the adjusted elevation difference within the tolerance limit of the new survey. 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 disturbance, is not considered a proper check. In addition, the survey is required to connect to any network control points within 3 km of its path. However, if the survey is run parallel to existing control, then the following table specifies the maximum spacing of extra connections between the survey and the control. At least one extra connection should always be made.

Distance, survey to network	*Maximum spacing of extra connections (km)*
0.5 km or less	5
0.5 km to 2.0 km	10
2.0 km to 3.0 km	20

Instrumentation

Order Class	First I	First II	Second I	Second II	Third
Leveling instrument					
Minimum repeatability of line of sight	0.25"	0.25"	0.50"	0.50"	1.00"
Leveling rod construction	IDS	IDS	IDS† or ISS	ISS	Wood or Metal
Instrument and rod resolution (combined)					
Least count (mm)	0.1	0.1	0.5-1.0*	1.0	1.0

(IDS—Invar, double scale)
(ISS—Invar, single scale)
† if optional micrometer is used.
* 1.0 mm if 3-wire method, 0.5 mm if optical micrometer.

Only a compensator or tilting leveling instrument with an optical micrometer should be used for first-order leveling. Leveling rods should be one piece. Wooden or metal rods may be employed only for third-order work. A turning point consisting of a steel turning pin with a driving cap should be utilized. If a steel pin cannot be driven, then a turning plate ("turtle") weighing at least 7 kg should be substituted. In situations allowing neither turning pins nor turning plates (sandy or marshy soils), a long wooden stake with a double-headed nail should be driven to a firm depth.

Calibration Procedures

Order Class	*First I*	*First II*	*Second I*	*Second II*	*Third*
Leveling instrument					
Maximum collimation error, single line of sight (mm/m)	0.05	0.05	0.05	0.05	0.10
Maximum collimation error, reversible compensator type instruments, mean of two lines of sight (mm/m)	0.02	0.02	0.02	0.02	0.04
Time interval between collimation error determinations not longer than (days)					
Reversible compensator	7	7	7	7	7
Other types	1	1	1	1	7
Maximum angular difference between two lines of sight, reversible compensator	40″	40″	40″	40″	60″
Leveling rod					
Minimum scale calibration standard	N	N	N	M	M
Time interval between scale calibrations (yr)	1	1	—	—	—
Leveling rod bubble verticality maintained to within	10′	10′	10′	10′	10′

(N—National standard)
(M—Manufacturer's standard)

Compensator-type instruments should be checked for proper operation at least every 2 weeks of use. Rod calibration should be repeated whenever the rod is dropped or damaged in any way. Rod levels should be checked for proper alignment once a week. The manufacturer's calibration standard should, as a minimum, describe scale behavior with respect to temperature.

Field Procedures

Order Class	*First I*	*First II*	*Second I*	*Second II*	*Third*
Minimal observation method	micro-meter	micro-meter	micro-meter or 3-wire	3-wire	center wire
Section running	SRDS or DR or SP	SRDS or DR or SP	SRDS or DR† or SP	SRDS or DR*	SRDS or DR§

Field Procedures—Continued

Order Class	*First I*	*First II*	*Second I*	*Second II*	*Third*
Difference of forward and backward sight lengths never to exceed					
per setup (m)	2	5	5	10	10
per section (m)	4	10	10	10	10
Maximum sight length (m)	50	60	60	70	90
Minimum ground clearance of line of sight (m)	0.5	0.5	0.5	0.5	0.5
Even number of setups when not using leveling rods with detailed calibration	yes	yes	yes	yes	—
Determine temperature gradient for the vertical range of the line of sight at each setup	yes	yes	yes	—	—
Maximum section misclosure (mm)	$3\sqrt{D}$	$4\sqrt{D}$	$6\sqrt{D}$	$8\sqrt{D}$	$12\sqrt{D}$
Maximum loop misclosure (mm)	$4\sqrt{E}$	$5\sqrt{E}$	$6\sqrt{E}$	$8\sqrt{E}$	$12\sqrt{E}$
Single-run methods					
Reverse direction of single runs every half day	yes	yes	yes	—	—
Nonreversible compensator leveling instruments					
Off-level/relevel instrument between observing the high and low rod scales	yes	yes	yes	—	—
3-wire method					
Reading check (difference between top and bottom intervals) for one setup not to exceed (tenths of rod units)	—	—	2	2	3
Read rod 1 first in alternate setup method	—	—	yes	yes	yes
Double scale rods					
Low-high scale elevation difference for one setup not to exceed (mm)					
With reversible compensator	0.40	1.00	1.00	2.00	2.00
Other instrument types:					
Half-centimeter rods	0.25	0.30	0.60	0.70	1.30
Full-centimeter rods	0.30	0.30	0.60	0.70	1.30

(SRDS—Single-Run, Double Simultaneous procedure)
(DR—Double-Run)
(SP—SPur, less than 25 km, double-run)
D—shortest length of section (one-way) in km
E—perimeter of loop in km
† Must double-run when using 3-wire method.
* May single-run if line length between network control points is less than 25 km.
§ May single-run if line length between network control points is less than 10 km.

Double-run leveling may always be used, but single-run leveling done with the double simultaneous procedure may be used only where it can be evaluated by loop closures. Rods should be leap-frogged between setups (alternate setup method). The date, beginning and ending times, cloud coverage, air temperature (to the nearest degree), temperature scale, and average wind speed should be recorded for each section plus any changes in the date, instrumentation, observer or time zone. The instrument need not be off-leveled/releveled between observing the high and low scales when using an instrument with a reversible compensator. The low-high scale difference tolerance for a reversible compensator is used only for the control of blunders.

With double scale rods, the following observing sequence should be used:

backsight, low-scale
backsight, stadia
foresight, low-scale
foresight, stadia
off-level/relevel or reverse compensator
foresight, high-scale
backsight, high-scale

Office Procedures

Order Class	*First* *I*	*First* *II*	*Second* *I*	*Second* *II*	*Third*
Section misclosures (backward and forward)					
Algebraic sum of all corrected section misclosures of a leveling line not to exceed (mm)	$3\sqrt{D}$	$4\sqrt{D}$	$6\sqrt{D}$	$8\sqrt{D}$	$12\sqrt{D}$
Section misclosure not to exceed (mm)	$3\sqrt{E}$	$4\sqrt{E}$	$6\sqrt{E}$	$8\sqrt{E}$	$12\sqrt{E}$
Loop misclosures					
Algebraic sum of all corrected misclosures not to exceed (mm)	$4\sqrt{F}$	$5\sqrt{F}$	$6\sqrt{F}$	$8\sqrt{F}$	$12\sqrt{F}$
Loop misclosure not to exceed (mm)	$4\sqrt{F}$	$5\sqrt{F}$	$6\sqrt{F}$	$8\sqrt{F}$	$12\sqrt{F}$

(D—shortest length of leveling line (one-way) in km)
(E—shortest one-way length of section in km)
(F—length of loop in km)

The normalized residuals from a minimally constrained least squares adjustment will be checked for blunders. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Elevation difference standard errors computed by error propagation in a correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models that account for:

gravity effect or orthometric correction
rod scale errors
rod (Invar) temperature
refraction—need latitude and longitude to 6" or vertical temperature difference observations between 0.5 and 2.5 m above the ground
earth tides and magnetic field
collimation error
crustal motion

3.6 Photogrammetry

Photogrammetry is a measurement system comprised of photographs taken by a precise metric camera and measured by a comparator. Photogrammetry is used for densification of horizontal control. The following specifications apply only to analytic methods.

Network Geometry

Order Class	*Second* *I*	*Second* *II*	*Third* *I*	*Third* *II*
Forward overlap not less than	66%	66%	60%	60%
Side overlap not less than	66%	66%	20%	20%
Intersecting rays per point not less than (design criteria)	9	8	3	3

The photogrammetric survey should be areal: single strips of photography are not acceptable. The survey should encompass, ideally, a minimum of eight horizontal control points and four vertical points spaced about the perimeter of the survey. In addition, the horizontal control points should be spaced no farther apart than seven air bases. The horizontal control points should have an order (and class) better than the intended order (and class) of the survey. The vertical points need not meet geodetic control standards. If the required control points are not available, then they must be established by some other measurement system.

Instrumentation

Order Class	*Second* *I*	*Second* *II*	*Third* *I*	*Third* *II*
Metric Camera				
Maximum warp of platen not more than (μm)	10	10	10	10
Dimensional control not less than	reseau with maximum spacing of 2 cm	8 fiducials	8 fiducials	8 fiducials
Comparator				
Least count (μm)	1	1	1	1

The camera should be of at least the quality of those employed for large-scale mapping. A platen should be included onto which the film must be satisfactorily flattened during exposure. Note that a reseau should be used for second-order, class I surveys.

Calibration Procedures

Order Class	*Second* *I*	*Second* *II*	*Third* *I*	*Third* *II*
Metric camera				
Root mean square of calibrated radial distortion not more than (μm)	1	3	3	5

Calibration Procedures—Continued

Order *Class*	*Second* *I*	*Second* *II*	*Third* *I*	*Third* *II*
Root mean square of calibrated decentering distortion not more than (μm)	1	5†	5†	5†
Root mean square of reseau coordinates not more than (μm)	1	1	3	3
Root mean square of fiducial coordinates not more than (μm)	—	1	3	3

† not usually treated separately in camera calibration facilities; manufacturer's certification is satisfactory.

The metric camera should be calibrated every 2 years, and the comparator should be calibrated every 6 months. These instruments should also be calibrated after repair or modifications.

Characteristics of the camera's internal geometry (radial symmetric distortion, decentered lens distortion, principal point and point of symmetry coordinates, and reseau coordinates) should be determined using recognized calibration techniques, like those described in the current edition of the *Manual of Photogrammetry*. These characteristics will be applied as corrections to the measured image coordinates.

Field Procedures

Photogrammetry involves hybrid measurements: a metric camera photographs targets and features in the field, and a comparator measures these photographs in an office environment. Although this section is entitled "Field Procedures," it deals with the actual measurement process and thus includes comparator specifications.

Order *Class*	*Second* *I*	*Second* *II*	*Third* *I*	*Third* *II*
Targets				
Control points targeted	yes	yes	yes	yes
Pass points targeted	yes	yes	optional	optional
Comparator				
Pointings per target not less than	4	3	2	2
Pointings per reseau (or fiducial) not less than	4	3	2	2
Number of different reseau intersections per target not less than	4	—	—	—
Rejection limit from mean of pointings per target (μm)	3	3	3	3

Office Procedures

Order *Class*	*Second* *I*	*Second* *II*	*Third* *I*	*Third* *II*
Root mean square of adjusted photocoordinates not more than (μm)	4	6	8	12

A least squares adjustment of the photocoordinates, constrained by the coordinates of the horizontal and vertical control points, will be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Distance standard errors computed by error propagation in this correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models that incorporate the quantities determined by calibration.

3.7 Satellite Doppler Positioning

Satellite Doppler positioning is a three-dimensional measurement system based on the radio signals of the U.S. Navy Navigational Satellite System (NNSS), commonly referred to as the TRANSIT system. Satellite Doppler positioning is used primarily to establish horizontal control.

The Doppler observations are processed to determine station positions in Cartesian coordinates, which can be transformed to geodetic coordinates (geodetic latitude and longitude and height above reference ellipsoid). There are two methods by which station positions can be derived: point positioning and relative positioning.

Point positioning, for geodetic applications, requires that the processing of the Doppler data be performed with the precise ephemerides that are supplied by the Defense Mapping Agency. In this method, data from a single station is processed to yield the station coordinates.

Relative positioning is possible when two or more receivers are operated together in the survey area. The processing of the Doppler data can be performed in four modes: simultaneous point positioning, translocation, semishort arc, and short arc. The specifications for relative positioning are valid only for data reduced by the semishort or short arc methods. The semishort arc mode allows up to 5 degrees of freedom in the ephemerides; the short arc mode allows 6 or more degrees of freedom. These modes allow the use of the broadcast ephemerides in place of the precise ephemerides.

The specifications quoted in the following sections are based on the experience gained from the analysis of Doppler surveys performed by agencies of the Federal government. Since the data are primarily from surveys performed within the continental United States, the precisions and related specifications may not be appropriate for other areas of the world.

Network Geometry

The order of a Doppler survey is determined by: the spacing between primary Doppler stations, the order of the base network stations from which the primaries are established, and the method of data reduction that is used. The order and class of a survey cannot exceed the lowest order (and class) of the base stations used to establish the survey.

The primary stations should be spaced at regular intervals which meet or exceed the spacing required for the desired accuracy of the survey. The primary stations will carry the same order as the survey.

Supplemental stations may be established in the same survey as the primary stations. The lowest order (and class) of a supplemental station is determined either by its spacing with, or by the order of, the nearest Doppler or other horizontal control station. The processing mode determines the allowable station spacing.

In carrying out a Doppler survey, one should occupy, using the same Doppler equipment and procedures, at least two existing horizontal network (base) stations of order (and class) equivalent to, or better than, the intended order (and class) of the Doppler survey. If the Doppler survey is to be first-order, at least three base stations must be occupied. If relative positioning is to be used, all base station base lines must be directly observed during the survey. Base stations should be selected near the perimeter of the survey, so as to encompass the entire survey.

Stations which have a precise elevation referenced by geodetic leveling to the National Geodetic Vertical Datum (NGVD) are preferred. This will allow geoidal heights to be determined. As many base stations as possible should be tied to the NGVD. If a selection is to be made, those stations should be chosen which span the largest portion of the survey.

If none of the selected base stations is tied to the NGVD, at least two, preferably more, bench marks of the NGVD should be occupied. An attempt should be made to span the entire survey area.

Datum shifts for transformation of point position solutions should be derived from the observations made on the base stations.

The minimum spacing, D, of the Doppler stations may be computed by a formula determined by the processing mode to be employed. This spacing is also used in conjunction with established control, and other Doppler control, to determine the order and class of the supplemental stations.

By using the appropriate formula, tables can be constructed showing station spacing as a function of point or relative one-sigma position precision (s_p or s_r) and desired survey (or station) order.

Point Positioning

$$D = 2\sqrt{2}\, s_p a$$

where

a = denominator of distance accuracy classification standard (e.g., a = 100,000 for first-order standard).

Order	*First*	*Second*	*Second*	*Third*	*Third*
Class		*I*	*II*	*I*	*II*
s_p (cm)			D (km)		
200	566	242	114	56	28
100	283	141	57	28	14
70	200	100	40	20	10
50	141	71	26	14	7

Relative Positioning

$$D = 2\, s_r a$$

where

a = denominator of distance accuracy classification standard (e.g., a = 100,000 for first-order standard).

Order	*First*	*Second*	*Second*	*Third*	*Third*
Class		*I*	*II*	*I*	*II*
s_r (cm)			D (km)		
50	100	50	20	10	5
35	70	35	14	7	4
20	40	20	8	4	2

However, the spacing for relative positioning should not exceed 500 km.

Instrumentation

The receivers should receive the two carrier frequencies transmitted by the NNSS. The receivers should record the Doppler count of the satellite, the receiver clock times, and the signal strength. The integration interval should be approximately 4.6 sec. Typically six or seven of these intervals are accumulated to form a 30-second Doppler count observation. The reference frequency should be stable to within $5.0(10^{-11})$ per 100 sec. The maximum difference from the average receiver delay should not exceed 50 μsec. The best estimate of the mean electrical center of the antenna should be marked. This mark will be the reference point for all height-of-antenna measurements.

Calibration Procedures

Receivers should be calibrated at least once a year, or whenever a modification to the equipment is made. It is desirable to perform a calibration before every project to verify that the equipment is operational. The two-receiver method explained next is preferred and should be used whenever possible.

Two-Receiver Method

The observations are made on a vector base line, of internal accuracy sufficient to serve as a comparison standard, 10 to 50 m in length. The base line should be located in an area free of radio interference in the 150 and 400 MHz frequencies. The procedures found in the table on relative positioning in "Field Procedures" under the 20 cm column heading will be used. The data are reduced by either shortarc or semishort arc methods. The receivers will be considered operational if the differences between the Doppler and the terrestrial base line components do not exceed 40 cm (along any coordinate axis).

Single-Receiver Method

Observations are made on a first-order station using the procedures found in the table on relative positioning in "Field Procedures" under the 50 cm column heading. The data are reduced with the precise ephemerides. The resultant position must agree within 1 m of the network position.

Field Procedures

The following tables of field procedures are valid only for measurements made with the Navy Navigational Satellite System (TRANSIT).

Point Positioning

s_p *(precise ephemerides)*	*50 cm*	*70 cm*	*100 cm*	*200 cm*
Max. standard deviation of mean of counts/pass (cm), broadcast ephemerides	25	25	25	25
Period of observation not less than (hr)	48	36	24	12
Number of observed passes not less than†	40	30	15	8
Number of acceptable passes (evaluated by on-site point processing) not less than	30	20	9	4
Minimum number of acceptable passes within each quadrant*	6	4	2	1
Frequency standard warm-up time (hr)				
crystal	48	48	24	24
atomic	1.5	1.5	1.0	1.0
Maximum interval between meteorological observations (hr)	6	§	§	§

† Number of passes refers to those for which the precise ephemerides are available for reduction.
* There should be a nearly equal number of northward and southward passes.
§ each setup, visit and takedown.

Relative positioning

s_r	*20 cm*	*35 cm*	*50 cm*
Maximum standard deviation of mean of counts/pass (cm), broadcast ephemerides	25	25	25
Period of observation not less than (hr)	48	36	24
Number of observed passes not less than†	40	30	15
Number of acceptable passes (evaluated by on-site point position processing) not less than	30	20	9
Minimum number of acceptable passes within each quadrant*	6	4	2
Frequency standard warm-up time (hr)			
crystal	48	48	48
atomic	1.5	1.5	1.5
Maximum interval between meteorological observations (hr)	6	6	§

† Number of observed passes refers to all satellites available for tracking and reduction with the broadcast or precise ephemerides.
* Number of northward and southward passes should be nearly equal.
§ Each setup, visit and takedown.

The antenna should be located where radio interference is minimal for the 150 and 400 MHz frequencies. Medium frequency radar, high voltage power lines, transformers, excessive noise from automotive ignition systems, and high power radio and television transmission antennas should be avoided. The horizon should not be obstructed above 7.5°.

The antenna should not be located near metal structures, or, when on the roof of a building, less than 2 m from the edge. The antenna must be stably located within 1 mm over the station mark for the duration of the observations. The height difference between the mark and the reference point for the antenna phase center should be measured to the nearest millimeter. If an antenna is moved while a pass is in progress, that pass is not acceptable. If moved, the antenna should be relocated within 5 mm of the original antenna height; otherwise the data may have to be processed as if two separate stations were established. In the case of a reoccupation of an existing Doppler station, the antenna should be relocated within 5 mm of the original observing height.

Long-term reference frequency drift should be monitored to ensure it does not exceed the manufacturer's specifications.

Observations of temperature and relative humidity should be collected, if possible, at or near the height of the phase center of the antenna. Observations of wet-bulb and dry-bulb temperature readings should be recorded to the nearest 0.5°C. Barometric readings at the station site should be recorded to the nearest millibar and corrected for difference in height between the antenna and barometer.

Office Procedures

The processing constants and criteria for determining the quality of point and relative positioning results are as follows:

1. For all passes for a given station occupation, the average number of Doppler counts per pass should be at least 20 (before processing).
2. The cutoff angle for both data points and passes should be 7.5°.
3. For a given pass, the maximum allowable rejection of counts, 3 sigma postprocessing, will be 10.
4. Counts rejected (excluding cutoff angle) for a solution should be less than 10 percent.
5. Depending on number of passes and quality of data, the standard deviation of the range residuals for all passes of a solution should range between:
   Point positioning—10 to 20 cm
   Relative positioning—5 to 20 cm

A minimally constrained least squares adjustment will be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Distance standard errors computed by error propagation between points in this correctly weighted least squares adjustment will indicate the maximum achievable accuracy classification. The formula presented in "Standards" will be used to arrive at the actual classification. The least squares adjustment will use models which account for:

- tropospheric scale bias, 10 percent uncertainty
- receiver time delay
- satellite/receiver frequency offset
- precise ephemeris
- tropospheric refraction
- ionospheric refraction
- long-term ephemeris variations
- crustal motion

3.8 Absolute Gravimetry

Absolute gravimetry is a measurement system which determines the magnitude of gravity at a station at a specific time. Absolute gravity measurements are used to establish and extend gravity control. Within the context of a geodetic gravity network, as discussed in "Standards," a series of absolute measurements at a control point is in itself sufficient to establish an absolute gravity value for that location.

The value of gravity at a point is time dependent, being subject to dynamic effects in the Earth. The extent of gravimetric stability can be determined only by repeated observations over many years.

Network Geometry

Network geometry cannot by systematized since absolute observations at a specific location are discrete and uncorrelated with other points. In absolute gravimetry, a network may consist of a single point.

A first-order, class I station must possess gravimetric stability, which only repeated measurements can determine. This gravimetric stability should not be confused with the accuracy determined at a specific time. It is possible for a value to be determined very precisely at two different dates and for the values at each of these respective dates to differ. Although the ultimate stability of a point cannot be determined by a single observation session, an attempt should be made to select sites which are believed to be tectonically stable, and sufficiently distant from large bodies of water to minimize ocean tide coastal loading.

The classification of first-order, class I is reserved for network points which have demonstrated long-term stability. To ensure this stability, the point should be reobserved at least twice during the year of establishment and thereafter at sufficient intervals to ensure the continuing stability of the point. The long-term drift should indicate that the value will not change by more than 20 μGal for at least 5 years. A point intended as first-order, class I will initially be classified as first-order, class II until stability during the first year is demonstrated.

Instrumentation

The system currently being used is a ballistic-laser device and is the only one at the current state of technology considered sufficiently accurate for absolute gravity measurements. An absolute instrument measures gravity at a specific elevation above the surface, usually about 1 m. For this reason, the gravity value is referenced to that level. A measurement of the vertical gravity gradient, using a relative gravity meter and a tripod, must be made to transfer the gravity value to ground level. The accuracy of the relative gravimeter must satisfy the gravity gradient specifications found in "Field Procedures."

Calibration Procedures

Ballistic-laser instruments are extremely delicate and each one represents a unique entity with its own characteristics. It is impossible to identify common systematic errors for all instruments. Therefore, the manufacturer's recommendations for individual instrument calibration should be followed rigorously.

To identify any possible bias associated with a particular instrument, comparisons with other absolute devices are strongly recommended whenever possible. Comparisons with previously established first-order, class I network points, as well as first-order, class II network points tied to the class I points, are also useful.

Field Procedures

The following specifications were determined from results of a prototype device built by J. Faller and M. Zumberge (Zumberge, M., "A Portable Apparatus for Absolute Measurements of the Earth's Gravity," Department of Physics, University of Colorado, 1981) and are given merely as a guideline. It is possible that some of these values may be inappropriate for other instruments or models. Therefore, exceptions to these specifications are allowed on a case-by-case basis upon the recommendation of the manufacturer. Deviations from the specifications should be noted upon submission of data for classification.

Order *Class*	*First* *I*	*First* *II*	*Second*	*Third*
Absolute measurement				
Standard deviation of each accepted measurement set not to exceed (μGal)	20	20	50	100
Minimum number of sets/ observation	5	5	5	5
Maximum difference of a measurement set from mean of all measurements (μGal)	12	12	37	48
Barometric pressure standard error (mbar)	4	4	—	—
Gradient measurement				
Standard deviation of measurement of vertical gravity gradient at time of observation (μGal/m)	5	5	5	5
Standard deviation of height of instrument above point (mm)	1	1	5	10

Office Procedures

The manufacturer of an absolute gravity instrument usually provides a reduction process which identifies and accounts for error sources and identifiable parameters. This procedure may be sufficient, making further office adjustments unnecessary.

A least squares adjustment will be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Gravity value standard deviations computed by error propagation in a correctly weighted, least squares adjustment will indicate the provisional accuracy classification. The least squares adjustment, as well as digital filtering techniques and/or sampling, should use models which account for:

atmospheric mass attraction
microseismic activity
instrumental characteristics
lunisolar attraction
elastic and plastic response of the Earth (tidal loading)

3.9 Relative Gravimetry

Relative gravimetry is a measurement system which determines the difference in magnitude of gravity between two stations. Relative gravity measurements are used to extend and densify gravity control.

Network Geometry

A first-order, class I station must possess gravimetric stability, which only repeated measurements can determine. This gravimetric stability should not be confused with the accuracy determined at a specific time. It is possible for a value to be determined very precisely at two different dates, and for the values at each of these respective dates to differ. Although the ultimate stability of a point cannot be determined by a single observation session, an attempt should be made to select sites which are believed to be tectonically stable.

The classification of first-order, class I is reserved for network points that have demonstrated long-term stability. To ensure this stability, the point should be reobserved at least twice during the year of establishment and thereafter at sufficient intervals. The long-term drift should indicate that the value will not change by more than the 20 μGal for at least 5 years. A point intended as first-order, class I will initially be classified as first-order, class II until stability during the first year is demonstrated.

The new survey is required to tie at least two network points, which should have an order (and class) equivalent to or better than the intended order (and class) of the new survey. This is required to check the validity of existing network points as well as to ensure instrument calibration. Users are encouraged to exceed this minimal requirement. However, if one of the network stations is a first-order, class I mark, then that station alone can satisfy the minimum connecting requirement if the intended order of the new survey is less than first-order.

Instrumentation

Regardless of the type of a relative gravimeter, the internal error is of primary concern.

Order *Class*	*First* *I*	*First* *II*	*Second*	*Third*
Minimum instrument internal error (one-sigma), (μGal)	10	10	20	30

The instrument's internal accuracy may be determined by performing a relative survey over a calibration line (see below) and examining the standard deviation of a single reading. This determination should be performed after the instrument is calibrated using the latest calibration information. Thus the internal error is the measure of instrument uncertainty after all possible systematic error sources have been eliminated by calibration.

Calibration Procedures

An instrument should be properly calibrated before a geodetic survey is performed. The most important calibration item is the determination of the mathematical model that relates dial units, voltage, or some other observable to milligals. This may consist only of a scale factor. In other cases the model may demonstrate nonlinearity or periodicity. Most manufacturers provide tables or scale factors with each instrument. Care must be taken to ensure the validity of these data over time.

When performing first-order work, this calibration model should be determined by a combination of bench tests and field measurements. The bench tests are specified by the manufacturer. A field calibration should be performed over existing control points of first-order, class I or II. The entire usable gravimeter range interval should be sampled to ensure an uncertainty of less than 5 μGal. FGCC member agencies have established calibration lines for this specific purpose.

The response of an instrument to air pressure and temperature should be determined. The meter should be adjusted or calibrated for various pressures and temperatures so that the allowable uncertainty from these sources does not exceed the values in the table below.

The manufacturer's recommendations should be followed to ensure that all internal criteria, such as galvanometer sensitivity, long and cross level or tilt sensitivity, and reading line, are within the manufacturer's allowable tolerances.

The response of an instrument due to local orientation should also be determined. Systematic differences may be due to an instrument's sensitivity to local magnetic variations. Manufacturers attempt to limit or negate such a response. However, if a meter displays a variation with respect to orientation, then one must either have the instrument repaired by the manufacturer, or minimize the effect by fixing the orientation of the instrument throughout a survey.

Order Class	First I	First II	Second	Third
Necessary for user to determine calibration model	Yes	Yes	Yes	No
Allowable uncertainty of calibration model (μGal)	5	5	10	15
Allowable uncertainty due to external air temperature changes (μGal)	1	1	3	—
Maximum uncertainty due to external air pressure changes (μGal)	1	1	2	—
Allowable uncertainty due to other factors (μGal)	3	3	5	—

Field Procedures

A relative gravity survey is performed using a sequence of measurements known as a loop sequence. There are three common types: ladder, modified ladder, and line.

The ladder sequence begins and ends at the same network point, with the survey points being observed twice during the sequence: once in forward running and once in backward running. Of course, more than one network point may be present in a ladder sequence.

Order Class	First I	First II	Second	Third
Minimum number of instruments used in survey	2	2	2	1
Recommended number of instruments used in survey	3	3	2	1
Allowable loop sequence	a	a	a,b	a,b,c
Minimum number of readings at each observation/instrument	5	5	2†	1
Standard deviation of consecutive readings (unclamped) from mean* not to exceed (μGal)	2	2	5	—
Monitor external temperature and air pressure	Yes	Yes	No	No
Standard deviation of temperature measurements (°C)	0.1	0.1	—	—
Standard deviation of air pressure measurement (mbar)	1	1	—	—
Standard deviation of height of instrument above point (mm)	1	1	5	10

(a—ladder) (b—modified ladder) (c—line)

† Although two readings are required, only one reading need be recorded.

* corrected for lunisolar attraction.

The modified ladder sequence also begins and ends at the same network point. However, not all the survey points are observed twice during the sequence. Again, more than one network point may be observed in the sequence.

The line sequence begins at a network point and ends at a different network point. A survey point in a line sequence is usually observed only once.

One should always monitor the internal temperature of the instrument to ensure it does not fluctuate beyond the manufacturer's recommended limits. The time of each reading should be recorded to the nearest minute.

Office Procedures

Order	First I	First II	Second	Third
Rejection Limits				
Maximum standard error of a gravity value (μGal)	20	20	50	100
Total allowable instrument uncertainty (μGal)	10	10	20	30
Model Uncertainties				
Uncertainty of atmospheric mass model (μGal)	0.5	0.5	—	—
Uncertainty of lunisolar attraction (μGal)	1	1	5	5
Uncertainty of Earth elastic and plastic response to tidal loading (μGal)	2	2	5	—

A least squares adjustment, constrained by the network configuration and precision of established gravity control, will be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Gravity standard errors computed by error propagation in a correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models which account for:

instrument calibrations
1) conversion factors (linear and higher order)
2) thermal response (if necessary)
3) atmospheric pressure response (if necessary)

instrument drift
1) static
2) dynamic

atmospheric mass attraction (if necessary)

Earth tides
1) lunisolar attraction
2) Earth elastic and plastic response (if necessary)

4. Information

Geodetic control data and cartographic information that pertain to the National Geodetic Control Networks are widely distributed by a component of the National Geodetic Survey, the National Geodetic Information Branch (NGIB). Users of this information include Federal, State, and local agencies, universities, private companies, and individuals. Data are furnished in response to individual orders, or by an automatic mailing service (the mechanism whereby users who maintain active geodetic files automatically receive newly published data for specified areas). Electronic retrieval of data can be carried out directly from the NGS data base by a user.

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 all horizontal control and approximately 65 percent of the vertical control. The remaining 35 percent are presented in the old formats; i.e., State leveling lines and description booklets. Until the old format data have been converted to the standard quadrangle formats, the vertical 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 Geodetic Control Networks are cartographically depicted on approximately 850 different control diagrams. NGS provides other related geodetic information: e.g., geoid heights, deflections of the vertical, calibration base lines, gravity values, astronomic positions, horizontal and vertical data for crustal movement studies, satellite-derived positions, UTM coordinates, computer programs, geodetic calculator programs, and reference materials from the NGS data bases.

The NGIB receives data from all NOAA geodetic 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 NGS data base. NOAA has entered into formal agreements with several Federal and State Government agencies whereby NGIB publishes, maintains, and distributes geodetic data received from these organizations. Guidelines and formats have been established to standardize the data for processing and inclusion into the NGS data base. These formats are available to organizations interested in participating in the transfer of their files to NOAA (appendix C).

Upon completion of the geodetic data base management system, information generated from the data base will be automatically revised. A new data output format is being designed for both horizontal and vertical published 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 automation system is fully implemented. Charges for digital data are determined on the basis of the individual requests, and reflect processing time, materials, and postage. The booklets *Publications of the National Geodetic Survey* and *Products and Services of the National Geodetic Survey* are available from NGIB.

For additional information, write:

Chief, National Geodetic Information
Branch, N/CG17
National Oceanic and Atmospheric Administration
Rockville, MD 20852

To order by telephone:

data:.. 301-443-8631
publications:.. 301-443-8316
computer programs or digital data:......... 301-443-8623

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(Special reference lists also follow appendixes A and B)

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APPENDIX A
Governmental Authority

A.1 Authority

The U.S. Department of Commerce's National Oceanic and Atmospheric Administration (NOAA) is responsible for establishing and maintaining the basic national horizontal, vertical, and gravity geodetic control networks to meet the needs of the Nation. Within NOAA this task is assigned to the National Geodetic Survey, a Division of the Office of Charting and Geodetic Services within the National Ocean Service. 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).

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, 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, 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 Bureau of the Budget classifications and standards.

The Federal Geodetic Control Committee (FGCC) 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).

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.

A.2 References

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APPENDIX B
Variance Factor Estimation

B.1 Introduction

The classification accuracies for the National Geodetic Control Networks measure how well a survey can provide position, elevation, and gravity. (More specifically, a distance accuracy is used for horizontal networks, and an elevation difference accuracy is used for vertical networks.) The interpretation of what is meant by "how well" contains two parts. A survey must be precise, i.e., fairly free of random error; it must also be accurate, i.e., relatively free of systematic error. This leads to a natural question of how to test for random and systematic error.

Testing for random error is an extremely broad subject, and is not examined here. It is assumed that the standard deviation of distance, elevation difference, or gravity provides an adequate basis to describe the amount of random error in a survey. Further, it is assumed that the selection of the worst instance of the classification accuracy computed at all points (or between all pairs of points) provides a satisfactory means of classifying a new survey. This procedure may seem harsh, but it allows the user of geodetic control to rely better upon a minimum quality of survey work. The nominal quality of a survey could be much higher.

Consider the method of observation equations (see Mikhail (1976) for a general discussion):

$$L_a = F(X_a)$$

where
L_a is a vector of computed values for the observations of dimension n,
X_a is a vector of coordinate and model parameters of dimension u, and
F is a vector of functions that describes the observations in terms of the parameters.

The design matrix, A, is defined as

$$A = \left.\frac{\partial F}{\partial X_a}\right|_{X_a = X_o}$$

where A is a matrix of differential changes in the observation model F with respect to the parameters, X_a, evaluated at a particular set of parameter values, X_o. A vector of observation misclosures is

$$L = L_b - L_a$$

where L_b is the vector of actual observations and L_a is the vector described above.

Associated with the observation vector L_b is a symmetric variance-covariance matrix Σ_{L_b}, which contains information on observation precision and correlation.

The observation equation may now be written in linearized form

$$AX = L + V$$

where V is a vector of residual errors and X is a vector of corrections to the parameter vector X_a. The least squares estimate of X is

$$X = (A^t(\Sigma_{L_b})^{-1}A)^{-1}\, A^t(\Sigma_{L_b})^{-1}L$$

where the superscripts ^{t} and $^{-1}$ denote transpose and inverse (of a matrix) respectively.

The estimate provides a new set of values for the parameters by

$$X_a + X \rightarrow X_a$$

If the observation model $F(X_a)$ is nonlinear (that is, A is not constant for any set of X_a), then the entire process, starting with the first equation, must be iterated until the vector X reaches a stationary point.

Once convergence is achieved, L_a, computed from the first equation, is the vector of adjusted observations. The vector of observation residual errors, V, is

$$V = L_a - L_b$$

Estimates of parameter precision and correlations are given by the adjusted parameter variance-covariance matrix, Σ_{X_a} computed by

$$\Sigma_{X_a} = (A^t(\Sigma_{L_b})^{-1}A)^{-1}.$$

The precision of any other quantity that can be derived from the parameters may also be computed. Suppose one wishes to compute a vector of quantities, S,

$$S = S(X_a)$$

from the adjusted parameters, X_a. A geometry matrix, G, is defined as

$$G = \left.\frac{\partial S}{\partial X_a}\right|_{X_a = X_o}$$

where G is a matrix of differential changes in the functions, S, with respect to the parameters, X_a, evaluated at a particular set of parameter values, X_o. By the principle of linear error propagation,

$$\Sigma_S = G\,\Sigma_{X_a} G^t$$

or

$$\Sigma_S = G(A^t(\Sigma_{L_b})^{-1}A)^{-1}\,G^t$$

where Σ_S is the variance-covariance matrix of the computed quantities.

This last equation is important since its terms are variances and covariances such as those for distance or height difference. Use of this equation assumes that the model is not too nonlinear, that the parameter vector X_a has been adequately estimated by the method of least squares, that the design matrix A, the geometry matrix G, and the variance-covariance matrix of the observations Σ_{L_b} are known. This last assumption is the focal point for the remainder of this appendix.

We must somehow estimate the n (n + 1)/2 elements of Σ_L. Usually, we know Σ_L subject to some global variance factor, f. We would then assume that

$$\Sigma_L = f\,\Sigma_L^0$$

where

Σ_L = the "true" variance-covariance matrix of the observations

Σ_L^0 = initial estimate of variance-covariance matrix of the observations

Our assumption about the the structure of Σ_L^0 relative to a single factor usually suffices. But this assumption can be improved if we generalize the idea. Consider a partition of the observations into k homogeneous groups. We now estimate k different local variance factors

$$\sum_L = \begin{pmatrix} f_1\Sigma_{L_1}^0 & & & 0 \\ & f_2\Sigma_{L_2}^0 & & \\ & & \ddots & \\ 0 & & & f_k\Sigma_{L_k}^0 \end{pmatrix}$$

As will be discussed later, we may also detect systematic error if one of the variance components is based on certified network observations.

B.2 Global Variance Factor Estimation (k = 1)

The global variance factor, f, is simply the a posteriori variance of unit weight, $\hat{\sigma}_0^2$, when given an a priori variance of unit weight, σ_0^2, equal to 1.

It can be shown that

$$E(V^t(\Sigma_L)^{-1}V) = n - u. \qquad \text{(Mikhail 1976: p. 287)}$$

For a single variance factor

$$\Sigma_L = f\,\Sigma_L^0$$

so that

$$\frac{1}{f}\Sigma(V^t(\Sigma_L^0)^{-1}V) = n - u$$

or for f to be unbiased (Hamilton 1964, p. 130)

$$f = \frac{E(V^t(\Sigma_L^0)^{-1}V)}{n-u} = \frac{V^t(\Sigma_L^0)^{-1}V}{n-u} \quad .$$

This is identical to the form $\hat{\sigma}_0^2 = \frac{V^tPV}{n-u}$, where P is defined as $\sigma_0^2(\Sigma_L^0)^{-1}$

Since we are given that $\sigma_0^2 = 1$, then $P = (\Sigma_L^0)^{-1}$. Then $f = \hat{\sigma}_0^2$, as we wished to prove.

The derivation assumes that there is no bias in the residuals (Mikhail 1976), i.e.,

$$E(V) = 0.$$

However, outliers, as well as systematic errors, can produce a biased global variance factor. We must be satisfied that the observations contain no blunders, and that our mathematical model is satisfactory in order to use the global variance factor.

Particular types of systematic errors—global scale or orientation errors—are not detectable in a survey adjustment. They will not bias the residuals and will not influence the global variance factor. For example, to detect a global scale error, it must be transformed into a local scale error by addition of more data or measurements that can discriminate between global and local.

B.3 Local Variance Factor Estimation (k = 2,3,...)

Let us separate our observations into k homogeneous groups, and assume that we know the variance-covariance matrices of all k groups, $\Sigma_{L_i}^0$, subject to k local variance factors, f_i. Then

$$\sum_L = \begin{pmatrix} f_1\Sigma_{L_1}^0 & & & 0 \\ & f_2\Sigma_{L_2}^0 & & \\ & & \ddots & \\ 0 & & & f_k\Sigma_{L_k}^0 \end{pmatrix}$$

A variety of methods has been proposed that can be used to estimate local variance factors. Among them are MInimum Norm Quadratic Unbiased Estimation (MINQUE) (Rao 1971), Iterated MInimum Norm Quadratic Estimation (IMINQE) (Rao 1972), Almost Unbiased Estimation (AUE) (Horn et al. 1975), and Iterated Almost Unbiased Estimation (IAUE) (Lucas 1984). Underlying these methods is the assumption that there is no bias in any group of residuals; that is

$$E(V_k) = 0.$$

This assumption can be turned to our advantage in the detection of local systematic error.

Consider the partition of observations into a network group, subscript N, and a survey group, subscript S ($k = 2$). Then

$$\Sigma_L = \begin{pmatrix} f_N \Sigma_N^0 & 0 \\ 0 & f_S \Sigma_S^0 \end{pmatrix}$$

For an adjustment of the network only, we may estimate

$$\Sigma_N' = f_N' \Sigma_N^0$$

and for an adjustment of the survey only, we may estimate

$$\Sigma_S' = f_S' \Sigma_S^0$$

where f'_S is the global variance factor of the survey observations computed by a least squares adjustment free of outliers and known systematic errors.

With perfect information and an unbiased model we compute $f_N = f'_N$ and $f_S = f'_S$. On the other hand, if our model is biased, this may not be the case. In other words, we have a linkage between systematic error and consistent estimation of local variance factors.

Now assume that our network observations are certified as having no systematic error, and that we have perfect knowledge of their weights. Then $f'_N = 1$ and $\Sigma_N = \Sigma_N^0$. In the absence of residual bias in the survey, we should compute $f_N = 1$ and $f_S = f'_S$. In fact, we could impose a constraint on the computation, $f_N = 1$, to ensure this result. A survey systematic error could then manifest itself as an increase in f_S over f'_S.

There is no guarantee that systematic error in a survey will increase f_S over f'_S. For example, a survey may be connected to the network at only one control point. A scale error local to the survey would remain undetectable with combined variance factor estimation. With a second connection to the network, the survey scale error will begin to be detectable. As the survey is more closely connected to the network, the capability to detect a survey scale error becomes much better. We see that systematic error in a survey that is well-connected to a certified geodetic network can be discovered by local variance factor estimation. Of course a systematic error, such as a scale factor influencing both the network and the survey, would continue to remain hidden.

B.4 Iterated Almost Unbiased Estimation (IAUE)

The IAUE method (Lucas 1984) can be used to estimate covariance elements as well as the variance elements of Σ_L. However, in testing for systematic error we are concerned only with the survey and the network variance factors ($k = 2$).

As suggested by the title, the method is iterative. We start with the initial values

$$f_S^0 \text{ and } \Sigma_S^0, \text{ with } f_N^0 \text{ set to } 1 .$$

Let

$$\Sigma_L^0 = \begin{pmatrix} f_N^0 \Sigma_N^0 & 0 \\ 0 & f_S^0 \Sigma_S^0 \end{pmatrix}$$

$$P_L^0 = (\Sigma_L^0)^{-1} = \begin{pmatrix} P_N^0 & 0 \\ 0 & P_S^0 \end{pmatrix}$$

We now iterate from $i = 0$ to convergence

1) Perform least squares adjustment for

$$\hat{X} = (A^t P_L^i A)^{-1} A^t P_L^i L .$$

2) $\Sigma_{V_S}^i = (P_S^i)^{-1} - A_S (A^t P_L^i A)^{-1} A_S^t .$

3) $$f_S^{i+1} = \frac{(V_S^i)^t P_S^i V_S^i}{tr(\Sigma_{V_S}^i P_S^i)}$$

where tr is the trace function.

4) $\Sigma_S^{i+1} = f_S^{i+1} \Sigma_S^i .$

We test for convergence by

$$\frac{f_S^{i+1} - f_S^i}{f_S^i} < \epsilon$$

where ϵ is a preset quantity > 0. The local survey variance factor is

$$f_S = \prod_{i=0}^{m} f_S^i$$

where m is the number of iterations to convergence. We can then compute a survey variance factor ratio,

$$f_S/f'_S$$

Computer simulations have shown that when the survey variance factor ratio exceeds 1.5, then the survey contains systematic error. This rule becomes less reliable when a survey is minimally connected to a network.

We note that for k = 1, the third step of the method yields

$$f^{i+1} = \frac{(V^tPV)^i}{n-u} \ .$$

It is immediately recognized as the a posteriori estimate of the variance of unit weight. In this special case, IAUE convergence is correct, immediate, and unbiased.

The IAUE method is particularly attractive from a computational point of view. If Σ_L is diagonal, or nearly so, then the requisite elements of Σ_L may be computed from elements of Σ_X that lie completely within the profile of the normal equations. Thus, the usual apparatus of sparse least squares adjustments can be retained.

B.5 References

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APPENDIX C

Procedures for Submitting Data to the National Geodetic Survey

The National Geodetic Survey (NGS) has determined that the value to the national network of geodetic observations performed by other Federal, State, and local organizations compensates for the costs of analyzing, adjusting, and publishing the associated data. Consequently, a procedure has been established for data from horizontal, vertical, and gravity control surveys to be submitted to NGS. Persons submitting data must adhere to the requirements stated herein, but in any event, the final decision of acceptance on data will be the responsibility of the Chief, NGS.

The survey data must be submitted in the format specified in the Federal Geodetic Control Committee (FGCC) publication, *Input Formats and Specifications of the National Geodetic Survey Data Base,* which describes the procedures for submission of data for adjustment and assimilation into the National Geodetic Survey data base. Volume I (Horizontal control data), volume II (Vertical control data) or volume III (Gravity control data) may be purchased from:

National Geodetic Information Branch (N/CG17x2)
National Oceanic and Atmospheric Administration
Rockville, MD 20852

Horizontal control surveys must be accomplished to at least third-order, class I standards and tied to the National Geodetic Horizontal Network. Vertical control surveys must be accomplished in accordance with third-order or higher standards and tied to the National Geodetic Vertical Network. Gravity control surveys must be accomplished to at least second-order standards and tied to the National Geodetic Gravity Network. Third-order gravity surveys ("detail" surveys) will be accepted by NGS for inclusion into the NGS Gravity Working Files only in accordance with the above mentioned FGCC publication. A clear and accurate station description should be provided for all control points.

The original field records (or acceptable copies), including sketches, record books, and project reports, are required. NGS will retain these records in the National Archives. This is necessary if questions arise concerning the surveys on which the adjusted data are based. In lieu of the original notes, high quality photo copies and microfilm are acceptable. The material in the original field books or sheets are needed, not the abstracts or intermediate computations.

Reconnaissance reports should be submitted before beginning the field measurements, describing proposed connections to the national network, the instrumentation, and the field procedures to be used. This will enable NGS to comment on the proposed survey, drawing on the information available in the NGS data base concerning the accuracy and condition of these points, and to determine if the proposed survey can meet its anticipated accuracy. This project review saves the submitting agency the expense of placing data that would fail to meet accuracy criteria into computer-readable form.

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