GEOMETRIC GEODETIC ACCURACY STANDARDS

AND

SPECIFICATIONS FOR USING GPS RELATIVE

POSITIONING TECHNIQUES

FEDERAL GEODETIC CONTROL COMMITTEE

Rear Adm. Wesley V. Hull, Chairman

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Note: This is a preliminary document. Use only as a guideline for the planning and execution of geodetic surveys using GPS relative positioning techniques.

FOREWORD

This document was first prepared and distributed in draft form as version 1.0 in May 1, 1985. It supersedes all previous versions, including version 4.0, dated September 1, 1986.

The document is subject to frequent revisions as requirements for classification of geodetic control surveys change, as the definitions for accuracy standards are modified, as we gain experience in performing GPS surveys with an enhanced satellite system, as GPS surveying equipment are improved, as the field procedures are streamlined, and as refinements are made to processing software.

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D I S C L A I M E R

Until this document is officially sanctioned by the Federal Geodetic Control Committee (FGCC), distribution does not constitute, in any way, an endorsement by the National Geodetic Survey, CGS, NOS, NOAA, or the FGCC. The "Geometric Geodetic Survey Standards and Specifications for Geodetic Surveys Using GPS Relative Positioning Techniques" is intended only for the purpose of providing the user, guidelines for planning, execution and classification of geodetic surveys performed by GPS satellite surveying relative positioning methods using carrier phase observations.

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ABSTRACT. The practical experience gained in performing relative positioning geodetic surveys using Global Positioning System (GPS) satellite surveying techniques, the advancements in software developments, improvements in geodetic survey receiver systems, development of improved planning methods and observing strategies, and the results of tests by the Federal Geodetic Control Committee, provide the basis for development of geometric (three-dimensional) geodetic survey standards and specifications for GPS relative positioning surveys. The geometric standards are designed to meet classification requirements for a wide range of three-dimensional relative positional accuracy requirements. The GPS specifications cover network geometry, instrumentation, calibration procedures, field procedures, and office reduction procedures. Because application of GPS relative positioning techniques are relatively new, definitions for the accuracy standards and specifications for field procedures and data analysis will undergo rapid evolution. This will mean frequent revisions for the next several years or until at least a few years beyond deployment of the Block II satellites of the operational GPS constellation (presently 1991).

INTRODUCTION

The rapidly growing use of the Global Positioning System (GPS) for geodetic surveying applications has resulted in a critical need for development of acceptable accuracy standards and GPS survey specifications. Specifications are essential to promote efficiency in the conduct of field operations and to facilitate classification of surveys.

The extensive GPS survey field experience and numerous reports on the analysis of results for GPS survey projects are the basis for developing specifications for GPS geodetic surveys. These results are documented in unpublished National Geodetic Survey (NGS) GPS project reports, published NGS special reports, the FGCC GPS survey system test reports, papers in the proceedings of the First International Symposium on Precise Positioning with the Global Positioning System held in Rockville, Maryland, April 1985 (Goad 1985), and papers in the proceedings of the Fourth International Geodetic Symposium on Satellite Positioning held in Austin, Texas, April 1986 (Defense Mapping Agency (DMA) and NGS 1986).

The GPS specifications are for control surveys performed by relative positioning techniques where two or more receivers are collecting carrier phase measurement data simultaneously. They are a guide for determining how to meet requirements for horizontal, vertical, and azimuth accuracy standards.

Survey standards are defined as minimum accuracies that are necessary to meet specific objectives. Specifications are defined as field methods required to meet a particular survey standard. This document will complement the FGCC (Federal Geodetic Control Committee) Standards and Specifications for Geodetic Control Networks dated September 1984 (FGCC 1984).

The 1984 standards for horizontal coordinates are based on a "distance accuracy standard" which is the ratio of the relative positional error of a pair of control points to the horizontal separation of those points. As this ratio increases, the classification of the control survey degrades. If a relative positional error is constant, classification degrades as minimum separation between stations decreases. Thus, there is a minimum station spacing for the 1984 standards. The most stringent distance accuracy standard is 1:100,000 (10 ppm) which is classified as an order 1 standard. For example, if the relative positional error was ± 1 cm (2 sigma), the minimum distance between stations in a project would be 1 km.

The 1984 vertical control standards, which are based on elevation difference accuracies, is considerably more stringent. For example, the maximum elevation difference accuracies for a first-order, class I, survey range from 2.0 to 0.05 ppm for bench marks spaced 1 to 100 km apart. This is computed using the equation $b = S/\sqrt{d}$ where b is the maximum elevation difference accuracy, S is propagated elevation difference in mm between stations, and d is distance between stations in km. Thus, for a relative positional error of ± 1 cm (2 sigma), the minimum distance between stations in a project would be 100 km.

Experience has shown that it is possible to successfully measure base lines by GPS relative positioning techniques and obtain precisions routinely at the (1 cm + 1-2 ppm) level in each component or 10 times better than the FGCC 1984 order 1 standard. With careful planning, the use of appropriate observing strategies, and data processing with optimized software and procedures, precisions approaching (0.3 cm + 0.01 ppm) have been achieved. This is 1000 times better than the existing order 1 distance accuracy standard of 1:100,000.

Geometric or ellipsoidal height differences, when combined with geoid height differences, can give very useful orthometric height differences. Typical accuracies for orthometric height differences determined from the results of GPS relative positioning surveys range from a centimeter to several decimeters (depending on location of the survey project and spacing between stations). In most cases, the dominant error in the orthometric height differences is the error in estimating the geoidal slope or geoid undulation differences (Zilkoski and Hothem 1988).

In part 1 of this document, geometric (three-dimensional) accuracy standards for classifying relative positioning surveys by space measurement techniques are presented. These accuracy standards complement the terrestrial distance accuracy standards provided in the September 1984 document. In addition to three low orders, three high order standards are provided: 0.01, 0.1 and 1 ppm.

To classify elevation differences determined indirectly from use of space survey systems such as GPS, accuracy standards consistent with expected user requirements are proposed in Appendix E. These proposed elevation difference standards do not replace the present FGCC accuracy standards for elevation differences determined directly by precise differential or trigonometric leveling measurement techniques. They are to be used only for classifying or specifying the accuracy for elevation (orthometric height) differences determined from systems that measure height differences relative to a reference ellipsoid rather than a mean sea level datum or the National Geodetic Vertical Datum (NGVD) 1929.

The format for GPS relative positioning specifications is based on the current edition of the FGCC document for standards and specifications for geodetic control networks (FGCC 1984). The section on specifications includes network design and geometry, instrumentation, calibration procedures, field procedures, and office processing procedures.

These geometric accuracy standards and GPS relative positioning survey specifications are now under review by the Federal Geodetic Control Committee (FGCC). The FGCC, a U.S. interagency committee, is officially responsible for the adoption of standards and specifications for geodetic control networks. (See appendix A.)

BACKGROUND

GPS satellite surveying is a three-dimensional measurement system based on observations of the radio signals of the NAVSTAR Global Positioning System. The GPS observations are processed to determine station positions in Cartesian coordinates (X,Y,Z), which can be converted to geodetic coordinates (latitude, longitude, and height-above-reference ellipsoid). With adequate connections to vertical control network points and determination of the height of the geoid, orthometric heights or elevations can be computed for the points with unknown elevations.

The present GPS system is made up of the Block I satellites. The Block II system of 21 to 24 satellites is expected to be in full operation by about 1991. There are three primary modes of access to the GPS satellite signals: the "Standard Positioning Service" (SPS), the "Precise Positioning Service" (PPS), and codeless. The SPS is based on the Course/Acquisition Code (C/A Code) for the L1 frequency only while the PPS will be based on access to the P-code for the L1 and L2 frequency. With the proposed encryption of the PPS for the Block II system allowing only restricted access, SPS and codeless may be the only options for most users. Receiver designs that incorporate codeless technology can observe the two frequencies without access to either the SPS or PPS codes. Another receiver design combines SPS tracking capability for the L1 signal and codeless technology for the L2 frequency.

There are two methods by which station positions can be derived: point positioning and relative positioning. In the point positioning method, data from a single station are processed to determine three-dimensional coordinates (X,Y,Z) referenced to the WGS-84 earth-centered reference frame (datum). The present accuracy for GPS point position determinations ranges between 50 cm to 10 m (one sigma) depending on the accuracy of the ephemerides and period of the observations.

To perform geodetic surveys at the decimeter-level or better, one must employ GPS relative positioning techniques. In relative positioning, two or more GPS geodetic receivers receive signals simultaneously from the same set of satellites. These observations are processed to obtain the components of the base line vectors between observing stations (station coordinate differences (dX,dY,dZ)).

When the coordinates for one or more stations are known, the coordinates for new points can be determined after adjusting for the systematic differences between the reference system for the GPS satellites and local geodetic network control.

The specifications in this document are presently limited to fixed or static mode of relative positioning survey operations. In the static mode receiver/antennas are not moving while data is being collected. Future versions of this document will include specifications for kinematic modes of operation where one or more receiver/antennas are moving (possibly stopping only briefly at survey points) while one or more other receivers are continuously collecting data at fixed locations.

Proposed selective availability (sa) and encryption restrictions should have very little or no effect on static relative positioning techniques.

Since January 21, 1987, the orbital coordinate data for the GPS satellites are computed in the World Geodetic System 1984 (WGS-84), an Earth-centered and Earth-fixed coordinate system (DMA 1987).

There are at least four GPS signal measurement types that have been used for relative positioning techniques: pseudorange, code phase, integrated Doppler, and carrier phase. Although these observables have different characteristics, they are all functions of the instantaneous ranges between satellite and ground stations and their time derivatives. The most precise measurement type is the carrier phase.

Carrier phase measurements are made by "beating" the satellite carrier signal with the signal from the local receiver oscillator. The frequencies of these signals differ, primarily, by the amount of the Doppler frequency. Carrier phase observations are measurements of the phase difference of received signals emitted by the satellite's oscillator and the nominal carrier signal generated by the receiver's oscillator (Remondi 1985). There are several receivers capable of measuring the carrier phase of the L1 signal (1575.42 MHz) and or both the L1 and L2 (1227.6 MHz) signals (McDonald et al. 1987).

There are numerous approaches to processing carrier phase measurements. They are generally referred to as single, double, triple, or undifferenced methods. Each can be designed for either single- or multi-baseline processing (Goad 1985, and DMA and NGS 1986). In the multiple base line data processing mode, the data are processed for a single observing session or for multiple observing sessions in a single adjustment. The multiple session mode is also called a network solution and is only practical if there are adequate links or common stations between the observing sessions.

The major factors affecting accuracy of relative position determinations in the static (land survey) mode are: accuracy of the satellite positions, capability to model atmospheric (ionospheric and tropospheric) refraction errors, receiver timing bias, and field procedural errors (Beutler et al 1987 and Kinlyside 1988). Although stable weather conditions should not degrade the results substantially, severe storm fronts passing over one or more of the survey sites during an observing session can substantially degrade the results. Development of methods and techniques to bring these error sources under control will enhance survey capability in terms of accuracy, logistics, and, therefore, economy.

The present estimated accuracy of the precise ephemeris for the GPS satellites is 1 part-per-million (ppm) or better. The accuracy of the broadcast ephemeris is estimated to be 2 to 3 ppm. When the GPS orbit coordinates are fixed in the data processing, the errors in the orbits will propagate proportionately into each component of the base line determinations. To obtain precise base line vectors at the 0.01 or 0.1 ppm level, the average allowable orbital errors will have to be much smaller than are presently available. Should such accuracies be required and the post-computed orbit is not accurate enough, then data from fiducial stations (continuous tracking stations) will be processed with the project's GPS observations. In this method, the satellite orbital coordinates are adjusted while simultaneously solving for the station coordinate differences.

STANDARDS

Classification Standards

Six "orders" of geometric relative positioning accuracy standards are specified. These are summarized in table 1. These standards, reflecting a wide range of accuracy requirements, augment the present distance accuracy standards found in the 1984 FGCC document (FGCC 1984). Potential uses or applications for each of the orders are included in the table. The accuracy standards at the 95 percent confidence level for the six orders range from a very stringent standard in centimeters of $\pm\sqrt{(0.3)^2+(0.1d0.01)^2}$ to $\pm\sqrt{(5.0)^2+(0.1d100)^2}$ for the lowest order (d is the vector baseline length in kilometers). The three highest accuracy orders are called AA, A and B, respectively.

The highly stringent accuracy standard of order AA has been achieved for projects where data was processed in conjunction with continuous tracking data collected at stations of the Cooperative International GPS Network (CIGNET). The data were processed using orbital adjustment techniques. The distances generally ranged between 500 to 5000 km. The orders for 1 and lower accuracy standards are comparable (except for exclusion of Order 3, Class II) to the orders provided in the FGCC September 1984 document. Thus, the standards are defined in reasonable conformance with present GPS surveying capabilities.

Although the concept of "order/class" is retained, it should not be used for specifying the accuracy for a survey and for final station classification purposes. The user of these standards should determine the real accuracy needs and the cost implications. The accuracy needs should be specified in terms of accuracy values in distance units and parts per million. In specifying the accuracy values, the range of distances between adjacent stations should be included. Given this information, appropriate procedures for meeting these specified standards can be proposed.

Table 1. -- Geometric relative positioning accuracy standards for three-dimensional surveys using space system techniques.

Survey categories	Order	(95 percent confidence level) Minimum geometric Accuracy standard		
		Base error	Line-length Dependent error	
		e (cm)	p (ppm)	a (1:a)
Global-regional geodynamics; deformation measurements	AA	0.3	0.01	1:100,000,000
National Geodetic Reference System, "primary" networks; regional-local geodynamics; deformation measurements	A	0.5	0.1	1: 10,000,000
National Geodetic Reference System, "secondary" networks; connections to the "primary" NGRS network; local geodynamics; deformation measurements; high-precision engineering surveys	B	0.8	1	1: 1,000,000
National Geodetic Reference System (Terrestrial based); dependent control surveys to meet mapping, land information, property, and engineering requirements	(C)			
	1	1.0	10	1: 100,000
	2-I	2.0	20	1: 50,000
	2-II	3.0	50	1: 20,000
	3	5.0	100	1: 10,000

Note: For ease of computation and understanding, it is assumed that the accuracy for each component of a vector base line measurement is equal to the linear accuracy standard for a single-dimensional measurement at the 95 percent confidence level. Thus, the linear one-standard deviation (s) is computed by:

$$s = \pm[\sqrt{e^2 + (0.1d \cdot p)^2}]/1.96. \quad \text{(See appendix B.)}$$

Where, d is the length of the baseline in kilometers. 5-26-88

In defining the accuracy standards, it was assumed that each component of the baseline determined by GPS relative positioning techniques are much alike, i.e. error sources that are highly correlated. Thus, no particular component has characteristics making it desirable to treat it differently from the other two components. It was also a premise that optimum accuracies achievable with GPS satellite surveying techniques are routinely and economically possible if the survey is carried out carefully and with adequate control of error sources.

The accuracy standards are not based on the technical training or ability of a surveyor, but instead they are based on the capabilities of the GPS measurement systems. As we approach the date when the Block II GPS satellites become fully operational, the cost of survey systems is expected to continue to decrease. Equipment costs as it relates to the economics of conducting a GPS survey will be an insignificant factor in determining overall project costs. Rather, the cost for a survey project will largely depend on costs for labor, logistical support, and other factors.

When specifying an accuracy standard for a survey there may be an "intended" standard that is substantially more stringent than a minimally "acceptable" accuracy standard. Today, GPS satellite geodetic survey systems (with carrier phase measurement capability) operated in the relative positioning static mode can yield vector baseline results with one-sigma uncertainties that are typically better than $\pm\sqrt{((1.0cm)^2+(0.1d2ppm)^2)}$ from data sets collected for periods of about 1 hour. Periods of less than 60 minutes can yield comparable results, but with lower reliability. Even with about 30 minutes of data consisting of 4 or more satellites, good geometric distribution, very few or no cycle slips, and an accurate ephemerides, it is possible to achieve results comparable to 60 minute data sets. Even though the present constellation is not optimized for getting reliable accuracies, the final classification for a GPS survey may still be within an "acceptable" standard.

In practice, scheduling the observing units to collect simultaneous data for less than 30 minutes can increase the risk of achieving unsuccessful observing sessions, particularly when there may be factors that would affect the quantity and/or quality of the observations. Furthermore, when operating in the static mode, the difference in operating costs between a 60 minute and 30 minute observing span is insignificant.

In developing the specifications for orders 1, 2, and 3, these orders were grouped with a single set of criteria. Thus, the specification criteria for design and field procedures were defined for four primary orders: AA, A, B and C (1, 2I, 2II, and 3). The only exception to this are the specifications for office procedures where a unique set of criteria was defined for each of the six orders.

There may be two "final" classifications for a GPS relative positioning survey project. The first, a "geometric" classification, would be determined by analysis of the internal consistency for a GPS relative positioning network. Data for this classification would be based on analysis of loop misclosures, repeat baseline results, and minimally constrained (free) least-squares network adjustments (independent of the local network control). The "geometric" classification is especially important for surveys that are designed to meet high-accuracy requirements such as for establishment of a high-precision primary networks, deformation measurement investigations (crustal motion, subsidence monitoring, motion of structures, etc.) and other special high precision engineering surveys.

The second classification for a GPS project would be based on the results of a constrained 3D adjustment where published coordinates for existing stations of the National Geodetic Reference System (NGRS) are either fixed or given weighted constraints. When a survey is adjusted into the local network control system, it would receive an "NGRS" classification that would depend on the accuracy of the existing horizontal network control. In the constrained adjustment, the existing network is "assumed to be correctly weighted and free of significant systematic error." The "NGRS" classification may also depend on the accuracy standard specified for the orthometric heights determined from the GPS relative positioning data. In turn, this would depend on the accuracy of the geoidal height differences.

Relative position accuracy denotes the relative accuracy of the various components between one station and other stations of a network. The concept of relative position accuracy can be applied to networks established by single-dimensional conventional measurements or by three-dimensional space system measurements. The accuracy standards in table 1 apply to both single-dimensional conventional terrestrial measurement techniques and three-dimensional GPS relative positioning techniques.

For each geometric relative position accuracy standard, the maximum allowable linear error in centimeters (at the 95 percent confidence level) can be computed for a corresponding station spacing by (see appendix B):

$$s = \sqrt{(e^2 + (0.1pd)^2)} \qquad (1)$$

where, s = maximum allowable error in centimeters at 95 percent confidence level
d = distance in kilometers between any two stations
p = the minimum geometric relative position accuracy standard in parts-per-million (ppm) at the 95 percent confidence level.
e = base error in centimeters (this includes station-dependent setup error)

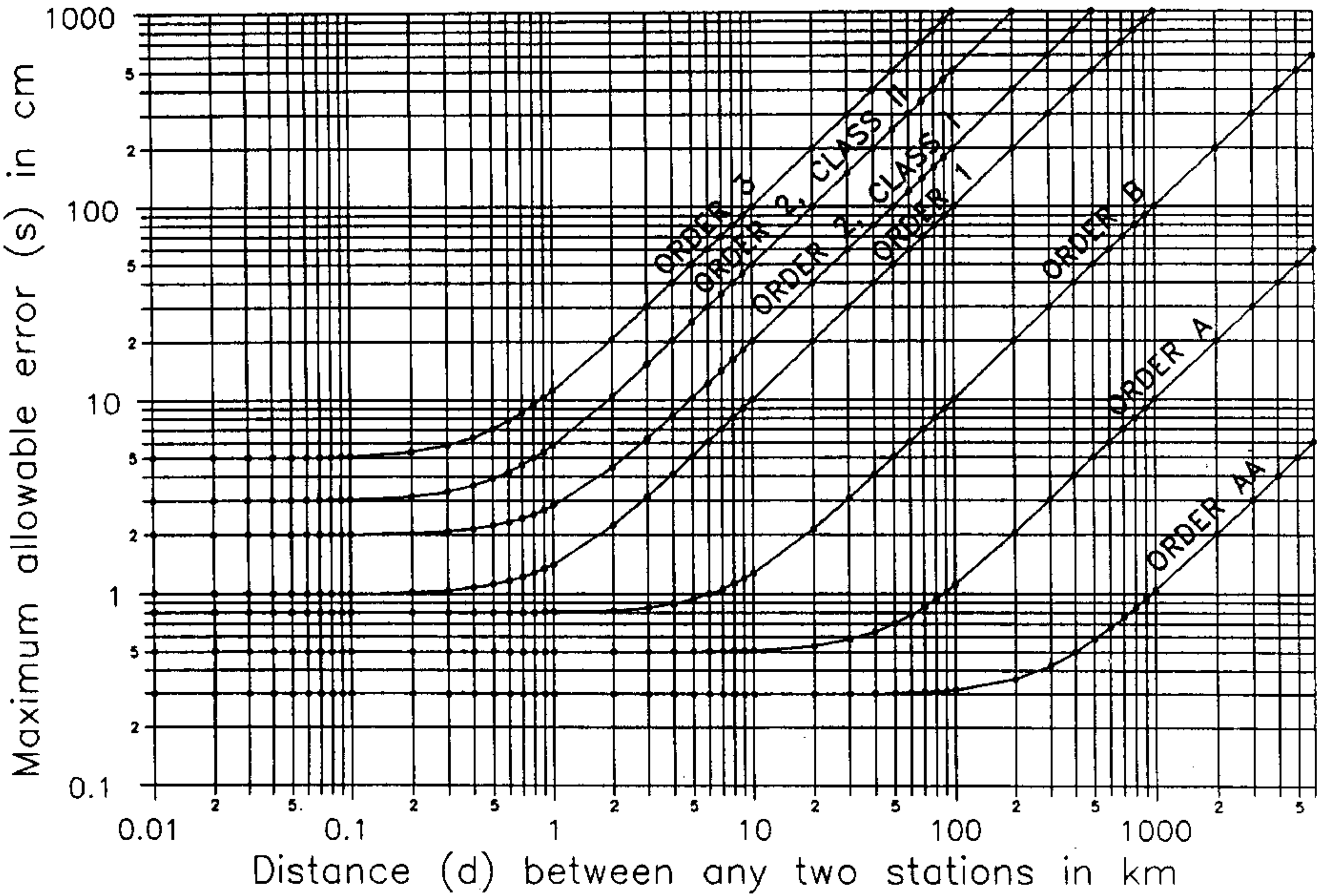
Figure 1 is a graph of the maximum spherical or linear error at the 95 percent confidence level for each order and class of the standards against the distance between any two stations.

Appendix C is a tabulation of one-sigma minimum standard errors computed from the minimum relative position accuracy standards given in table 1.

A survey station of a network is classified according to whether the propagated error at the 95 percent confidence region is less than or equal to the maximum allowable error "s" specified for the project. In the case of GPS determined baseline vectors, typically, the error propagation proceeds linearly for distances greater than about 20 km. The magnitude of the line-length dependent error will depend on the quality and quantity of the observations and the effectiveness of the baseline processing software for minimizing linearly dependent error sources.

For example, two stations are spaced 10 km apart and the accuracy standard for the baseline measurement is specified as order 1. The maximum allowable geometric relative error (at the 95 percent confidence level) between stations 1 and 2 is 10 cm. In this example, the value for s of 10 cm is 10 times greater than the base error of 1 cm. Thus, the base error (e) does not contribute significantly to the total value for s. On the other hand, if order B is specified, s = 1.3 cm. In this case, s is less than a factor of 2 greater than the value for e, thus the base error e is significant.

Geometric relative positioning accuracy standards
Maximum allowable error at 95% confidence level



This shows the importance of taking extra precautions to minimize the contribution to the base error caused by problems with antenna setup, antenna phase center stability, and signal multipath.

The minimum geometric relative position accuracies in table 1 represent present capabilities for making GPS baseline measurements. This includes any significant errors due to antenna setup (plumbing or centering, and measurement of height of antenna phase center above the station mark). The setup error can be the dominant error when establishing closely spaced stations for any of the accuracy standards. It may be the most significant error source when measuring widely spaced stations at the high accuracy orders. To control this potentially significant error source, a range of setup errors for corresponding accuracy standards and distances between stations are presented in appendix D. The errors were computed using a factor of 0.05 for the critical region (100 minus 95 percent confidence level). The setup error (k) in each component (N,E,U) at the 95 percent confidence level can be computed from:

$$k = 0.1pd(0.05), \text{ where, } k_{min} = 0.3 \text{ cm, and } k_{max} = 10 \text{ cm.}$$

The value for k_{min} is based on current realistic estimates for expected setup errors. The value for k_{max} is a worst-case setup error; in practice, it should be much smaller than 10 cm, typically less than 1 cm.

Although the accurate measurement of geometric quantities is important, in practice, orthometric heights or elevations may be desired in addition to ellipsoid heights. In many areas, the geoid slopes are usually less than most required accuracies for the orthometric height differences. For example, in most areas of the conterminous US, the slopes are well within 25 ppm. However, in some areas, such as mountainous regions, it might exceed 75 ppm. This may not be tolerable except for very low elevation difference accuracy standards. Most applications requiring either geometric order AA or A are concerned with changes with time rather than spatial differences, and hence are not sensitive to the difference between orthometric and ellipsoid heights since the two will generally change together in time (Kaula 1986).

In consideration that standards of accuracies for vertical control by spirit leveling should be different from those by GPS relative positioning and other 3-D geometric techniques. In appendix E, elevation (orthometric height) difference accuracy standards for geometric relative positioning techniques are proposed. The minimum accuracies for the geoid height differences that are required to achieve the desired elevation difference accuracy standard are also given. This, in effect, separates the accuracy standard for allowable geometric relative positioning error from the accuracy standards for elevation differences.

Specifications for a survey might include only a geometric accuracy standard but not an elevation difference accuracy standard. For example, one might perform a purely geometric survey if primarily interested in changing geometry such as plate motion investigations, subsidence monitoring, dam deformation studies, etc. The geometric relative position measurements can be evaluated to meet these high-precision purposes independent of the geoid.

In summary, the heights produced from GPS surveys are with respect to a reference ellipsoid. To convert these ellipsoid (also known as geodetic) heights to orthometric heights or elevations, the survey must include adequate connections to network control points with orthometric heights established by differential leveling techniques and referenced to the National Geodetic Vertical Datum (NGVD). When reliable estimates for the geoid height differences between all stations of the project are available, orthometric heights derived from the GPS survey results can be computed. The accuracy of the GPS derived orthometric heights will depend on the accuracy of the GPS ellipsoidal height differences, the accuracy of the orthometric heights for the vertical control, and the accuracy of geoid height differences.

The maximum azimuth accuracy from GPS relative position determinations is based on a minimum spacing between a pair of stations that are intervisible. The azimuth between a station pair is determined after adjustment of the vector baseline in the satellite reference system to the local datum reference system. For a specified azimuth accuracy and expected accuracy for the GPS vector baseline determinations at the 95 percent confidence level, a minimum spacing between a pair of stations can be computed. More discussion on azimuth determinations from GPS relative positioning surveys is contained in the next section.

These accuracy standards were developed in consideration of a critical need for statistically-based positional accuracy standards that is appropriate for three-dimensional measurement techniques such as use of the GPS. There is also recognized that statistically-based positional accuracy standards need to be developed for property and cadastral surveys (Vonderohe 1986). In Vonderohe's paper, it was indicated that relative error ellipses may be viable as standards. There is also a critical need for positional accuracy standards when making deformation measurements (monitoring vertical and horizontal movement) or for other precise engineering surveying applications.

As noted by Vonderohe (1986), discussion of positional accuracy standards should consider the practicality of users implementing them. The use of these standards requires a fundamental understanding of statistics and adjustments. But these educational requirements are not unique for the implementation of these standards. If the surveyor wants to help ensure successful use of GPS surveying techniques in a variety of applications, it would be prudent to acquire appropriate knowledge in statistics, adjustments, and analysis of observations.

Research or studies into the appropriate definition of statistically-based positional accuracy standards is clearly needed. Thus, as such research or studies bear new information, modification and refinements of these geometric accuracy standards are expected during the next few years.

Monumentation

With the increasing use of space system measurement techniques, such as use of GPS, it is important that station markers have the properties of permanence and stability. The markers must be stable in all three dimensions.

Factors that may affect the stability of a monument include frost heave action, changes in groundwater level, and settlement (Sliwa 1987). When selecting sites for stations of a high-precision primary network or for monitoring deformation, it is recommended that soil and geotechnical specialists be consulted.

Markers for existing network control should show no historical evidence of significant movement. If an existing network control marker does not exhibit adequately the properties of permanence and stability, it may have to be replaced by a new marker. The decision to replace old markers will depend on there use and purpose in future surveys.

The type of marker best suited for a given type or condition of terrain will depend on such factors as local conditions, transportation, materials available, equipment available for setting marks, and cost. Sites for new markers will, whenever possible, be located on public property such as road right-of-ways, public building grounds, school yards, etc.

To meet the requirements for permanent and stable monumentation, the markers are usually corrosion-resistant metal disks that may be set in a rock outcrop or large masses of concrete such as bridge abutments and other structural foundations.

When bedrock or large, massive structures are not available, it is more difficult to ensure the marker has the properties of permanence and stability. Traditional concrete monuments, with or without an underground mark, are not recommended as a suitable choice for preserving the three-dimensional coordinates.

The recommended alternative is a three-dimensional rod mark (Beard 1986). The principle component of the mark is a 9/16-inch stainless steel rod driven into the ground until the driving rate with a gasoline powered reciprocating hammer slows to 60 seconds per foot or slower. When in position, the top of the rod is just below ground level. The top of the rod is rounded and centerpunched, to mark the exact point to be positioned.

A grease-filled, 1-inch PVC pipe (sleeve) surrounds the rod from just below its top to a depth of at least 3 feet. It is preferable that the sleeve depth is equal to the depth of maximum frost penetration. Extreme depths of frost penetration for the conterminous U.S. is shown in figure 2. A hole must be dug for the sleeve during installation. The 1-inch sleeve reduces vertical stress to the rod caused by frost heave or other soil movements. It also helps restrict horizontal movement to an insignificant amount. The grease used to fill the sleeve should be an insoluble, non-corrosive, cold-weather type such as that conforming to U.S. military specification G-10924D. The grease is contained within the sleeve with pipe caps center drilled to 9/16 inch + 0.005 inch, allowing the rod to penetrate.

A 5-inch PVC pipe and cap with access cover is placed in concrete around the top of the assembly for protection and to aid in locating the mark. It is installed at or slightly above ground level The space between the 1-inch and 5-inch PVC pipe is filled with fine grain sand. (See appendix H for detailed setting procedures.)

When the sites for new points are being selected, surveyors should attempt to locate the new points on existing bench marks tied to the National Geodetic Vertical Network. Besides being prudent and cost-saving, this procedure will help

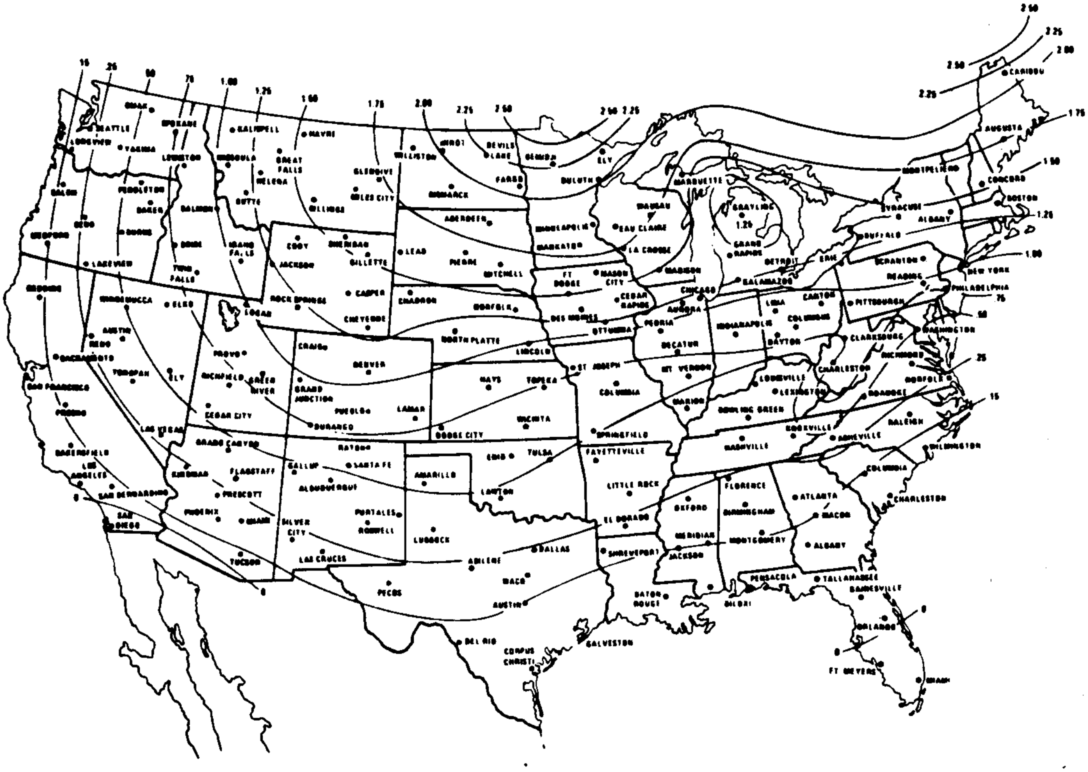


Figure 2.--Extreme depth of frost penetration (in meters) for conterminous U.S.

meet the requirements for connecting the markers with unknown elevations to the existing vertical network control. Should the permanency and/or stability of the bench mark be questionable, an offset marker may need to be set.

Reference marks are optional except in special circumstances. These circumstances could include: stations established for the National Crustal Motion Network, the primary National Geodetic Reference System, or other precise geodetic applications where recovery of a primary station is important for historical or legal reasons.

Whenever it is not possible to occupy a station directly and an offset point must be established, the offset point will be monumented and connected to the control station by survey techniques consistent with the accuracy standard specified for the GPS survey.

When practical, new stations should be located at sites that are accessible by ground transportation.

SPECIFICATIONS

The specifications recommended in the following sections are based on considerable practical experience. Some of the parameters may still reflect conservation estimates and will require further studies before they can be refined.

Development of the specifications is an evolutionary process that is not expected to stabilize before 1992 or after the Block II constellation of GPS satellites are launched and fully operational. Appendix J summarize the proposed Launch Dates, for the Block II GPS Satellites as of February 1988.

Network Design, Geometry, and Connections

The location and relative disposition of the control points do not depend significantly on factors such as network shape or intervisibility (except when establishing azimuth reference points) but rather on optimum layout for carrying out the intent of the survey.

Table 2 summarizes the specifications for the network design and connection factors, including minimum station spacing, ties to existing horizontal and vertical network control points, and direct connection requirements.

Checks should be made to ensure that no existing network control points have been moved or disturbed. It may be necessary to occupy more than the minimum number of network control points to ensure the survey is tied into points with sufficient accuracy or internal consistency.

If bench marks are located in areas subjected to vertical motion, it may be necessary to perform a vertical survey by differential or precise trigonometric leveling methods to ensure all bench marks are connected to a common epoch.

It is stated in the present FGCC specifications that whenever the distance between two unconnected survey points is less than 20 percent of the distance between those points traced along existing or new connections, a direct connection should be made between those survey points (FGCC 1984). The enforcement of this rule is optional depending on circumstances for stations located within the area of the GPS survey project.

At least three factors should be considered when determining whether direct connections between adjacent stations is desirable: (1) if an existing station, can it be recovered, (2) is the station reasonably accessible (i.e., it is quite likely it may be occupied during future surveys), and (3) what is the distance between the adjacent stations? When direct connections are desirable, table 2 provides guidelines for corresponding accuracy standards. If enforcement of the 'adjacent-station' rule is not practical, appropriate statements about those stations affected must be included in the project report.

If azimuth marks are required, the azimuth reference can be established by GPS surveys. There are at least four factors to consider when establishing azimuth references by GPS relative positioning techniques rather than using conventional

Table 2. -- Guidelines for network design, geometry and connections

Geometric accuracy standards	Group	AA	A	B	C
	Order	AA	A	B	1,2-I&II,3
	ppm	0.01	0.1	1.0	10,20,50,100
	base(cm)	0.3	0.5	0.8	1 2 3 5
Horizontal network control of NGRS[a], minimum number of stations:					
When connections are to orders AA, A or B....		4	3	3	2
When connections are to order 1.............		na[b]	na[b]	na[b]	3
When connections are to orders 2 or 3........		na[b]	na[b]	na[b]	4
Vertical network control of NGRS[a], minimum number of stations[c][d]..........		5	5	5	4
Continuous tracking stations (master or fiducials), minimum number of stations....		4	3	2	op
Station Spacing (km):					
Between "existing network control" and CENTER of project:					
Not more than.........................		100d	10d	7d	5d
50 percent not less than...............		$\sqrt{5d}$	$\sqrt{5d}$	$\sqrt{5d}$	d/5
Between "existing network control" located outside of project's outer boundary and the edge of the boundary, not more than.		3000	300	100	50
Location of network control (relative to center of project); number of "quadrants", not less than.............		4	4	3	3
Direct connections should be performed, if practical, between: ANY adjacent stations (new or old, GPS or non-GPS) located near or within project area, when spacing is less than (km)..........		30	30	10	5

Legend: d - is the maximum distance in (km) between the center of the project area and any station of the project.
NGRS - National Geodetic Reference System
CL - Confidence level; na - not applicable; op - optional

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Table 2. -- Guidelines for network design, geometry and connections (continued)

NOTE: If it is not practical to plan a survey that is within the criteria, minor adjustments may be made provided that it is authorized by the agency requesting the survey.
Remarks: (a) Consult National Geodetic Survey officials whenever it is necessary to consider exceptions to these criteria, particularly, when the GPS survey project data are to be submitted to NGS for incorporation in the NGRS.
(b) If a survey with an accuracy standard of AA, A, or B is specified and one objective in the survey is to upgrade the existing network, then connections to a minimum of four stations are required or at least one station in each one-degree block with a minimum of four stations.
(c) First choice is vertical network control established and/or maintained by the National Geodetic Survey. When it is not possible to occupy the minimum number of NGRS points, non-NGRS control points may be used. This should be documented in the project report.
(d) If it is expected that the constrained adjustment for determination of the elevations within the project area will be based on more than one "bias group" (see discussion under section on Office procedures, Analysis and Adjustments) then the minimum number of stations specified is that which is required within the area for each "bias group." For example, if there two bias groups and ties required to four bench marks, then four bench marks will be incorporated within each area of the "bias group" for a total of 8 bench marks.

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astronomical methods. They are: (1) cost, (2) a pair of stations will be located close to each other with coordinates established at the same order of accuracy, (3) repeat observations between the azimuth and main station can be used to verify the relative stability of the two marks in all three dimensions, and (4) check observations or redundancy is not possible when azimuth reference is determined from only a single set of astronomic observations.

Table 3 summarizes minimum spacings between station-pairs for corresponding relative position accuracies possibly achieved from a GPS survey and for a range of azimuth accuracy standards.

Table 3. -- Guidelines for minimum spacings for establishing pairs of intervisible stations to meet azimuth reference requirements.

Spacing between a "pair" of stations, not less than (meters)	Azimuth accuracy required in seconds of arc (95 percent confidence level) 1	2	4	6	10
	GPS relative position precision (mm) (95 percent confidence level)				
100	-	-	2	3	5
200	-	2	4	6	10
300	-	3	6	9	14
400	2	4	8	12	19
500	3	5	10	14	24
600	3	6	12	18	29

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Example: If the expected relative position precision from a GPS survey between two marks spaced less than 1000 meters apart is 2 mm at the 95 percent confidence level, then to achieve an azimuth accuracy of 2 seconds at the 95 percent confidence level, the minimum spacing between the pair of stations is 200 meters.

Instrumentation

GPS geodetic receivers may receive one or both carrier frequencies transmitted by the GPS satellites. Two frequency receivers are required for the most precise surveys to correct for the effects of ionospheric refraction where the magnitude of the error may range from 1 to 10 ppm. The receivers must record the phase of the satellite signals, the receiver clock times, and the signal strength. Data collected with different receivers may be combined in the processing, however, observations need to be taken approximately simultaneously.

Generally, GPS satellite geodetic surveying equipment will consist of three major components: the antenna, receiver/processor, and recording unit. Depending on type of cable used, the lengths will usually range from 10 to 60 meters. The maximum length and type of cable may depend on the manufacturer's specifications. The receiver should have the capability to track a minimum of four GPS satellites.

Some receivers may have multiple data ports for handling printer output, data input from automatic weather instruments, and remote control operations. It should be possible to operate the receiver in the unattended mode. However, when commanded, information should be available for display to ensure that the receiver is functioning normally and the data quality meets acceptable standards.

The receivers may be codeless or have the capability to receive and decode the P and/or CA coded data. If it is codeless, the receiver must have the appropriate output and input ports for synchronizing the clocks among instruments and with respect to UTC (Universal Coordinated Time).

The required stability of the reference frequency of the GPS receiver is dependent on the receiver design. The amount of the initial time offset between receivers and the relative drift which can be tolerated is highly dependent on the sophistication of the processing software (i.e., the physical model). All GPS receivers should have a signal input port for an external frequency standard.

For high precision results, while allowing the widest choice of processing software, it is recommended that codeless receivers be initially synchronized and the relative drift rates be maintained to less than 10 microseconds per hour. (This is equivalent to approximately 4.4 Hz difference in the GPS receiver's L1 frequency.) It is generally recommended that codeless receivers be compared again at the end of the surveying day. This is not strictly required; it is possible to perform the clock check the following day prior to synchronization.

For codeless sets it is recommended that a high quality wrist watch be standard equipment. In rare cases, the receiver clock may experience a time problem on the way to or at the survey site. In such a case a synchronization of the receiver clock to the wrist watch will likely result in a successful survey. The final processed results may be somewhat degraded, however.

The height of the "phase" center (L_1) or centers (L_1 and L_2) above a defined reference point on the antenna or an adaptor connected to the antenna is usually predetermined by the manufacturer. This will be a constant for a particular antenna model. Combining this height constant with the height of the defined reference point above the station mark will give the total height used to reduce the baseline measurements from phase-center to phase-center down to mark-to-mark. The location of the phase center may not be marked on the antenna.

Using the appropriate constant for a particular antenna model is very important when different antennas are used during the same project. If the bias in height between different antenna models is not well known, it is recommended that test surveys be conducted between nearby marks which have accurately known height differences. Then the constant for one of the antennas will be adjusted for any significant height bias between different antenna models.

Calibration

Field calibration is necessary to control systematic errors that may be critical to GPS satellite surveys. This will verify the adequacy of the GPS survey equipment, observation procedures, the processing software, and steps implemented in the data analysis. The field calibration consists of testing the GPS equipment performance and the associated base line processing software on a three-dimensional test network.

The three-dimensional test network should be composed of four or more stations spaced approximately 50 m to 10 km apart. The location of the stations should permit base lines to be measured which are nearly at right angles to each other.

Three-dimensional relative position measurements will be established to be accurate in any component to within $\pm\sqrt{((3mm)^2+(0.1d1ppm)^2)}$ at the 95 percent confidence level.

The field procedures found in table 4 for order B will be used to establish the test network. The data will be reduced in the fixed orbit mode using precise ephemerides available from the National Geodetic Survey (Remondi 1986). Single base line, multiple base line (session) processing software, or other software that will give results with comparable precision shall be used. The network shall be established with a minimum of four receivers collecting three observing days.

A special three-dimensional geodetic test network established by the FGCC has been used to test GPS survey systems since 1983. This network is located in the vicinity of Washington, D.C. (Hothem and Fronczek 1983).

If different receivers and/or different model antennas are used in a survey, it will be necessary to conduct calibration tests to determine whether significant biases exist. For example, if the markings for the location of the phase center are not at the true location for different antennas, this will cause a bias in the height component of the GPS base line measurements. Other tests may be needed to determine procedures to ensure optimum orientation of the antenna and to determine the error contribution due to multipath.

Field Procedures

The precision of the GPS vector base line results depends on the number of satellites visible simultaneously from each station during an observing session, their geometric relationships, duration of the period when the desired number of satellites can be observed simultaneously, the uncorrected effects of ionospheric and tropospheric refraction, and the length of line. The number of possible observing sessions per observing day is a function of the required survey accuracy, satellite availability, and project logistical considerations such as travel and set up time required between observing sessions.

The specifications for field procedures will be common for all surveys with "intended" accuracies specified as 1:100,000 or lower. This is because vector base lines can be measured routinely with uncertainties of better than 10 ppm (1:100,000) using data sets from collection periods of 30 to 60 minutes. Even data collection periods of a few minutes can also produce good results during optimal satellite visibility conditions.

Although there are no differences in the field procedures for 1:100,000 and lower order surveys, there will be different criteria for each standard in the section on office procedures. The criteria for establishing the "final" classification will differ significantly to take into account factors which affected the results and either were not known at the time the observations were being collected or they could not be controlled by altering the field procedures.

Factors possibly affecting the results include: unexpected degraded accuracy for the orbital coordinates, satellite transmission problems, significant atmospheric disturbances, and receiver problems that went undetected before the survey team departed from the project area. It will be possible from the office procedures to evaluate surveys affected by unexpected problems and determine a final classification that, although maybe lower than the "intended" accuracy, may still meet minimum criteria for a project.

Currently, the Block I GPS satellite constellation includes only seven usable satellites. Depending on the location of the project, this limits the observing period when four or more satellites are available to approximately 5 hours each day. When the Block II 21- to 24-satellite constellation becomes operational in the 1990's (See appendix I), in general at least six satellites will be available for simultaneous observations from anywhere on Earth 24 hours a day.

Table 4 summarizes the field procedures that should be followed to achieve the desired accuracy standards. These field procedures are valid only for relative positioning surveys and are subject to change as more satellites become available and processing techniques are refined.

Although there has not been any report of interference affecting quality of data, it advisable that the antenna be located where potential radio interference is minimal for the 1227.6 and 1575.42 MHz frequencies (GPS L1 and L2 signals). The distance between the potential radio interference and the GPS survey system may be an important consideration. For example, stations located adjacent to high-powered radio and high frequency, high-powered radar and transmission antennas should be avoided.

If one or more of the stations in a project network is continuously reoccupied during each session, these stations are generally called "master" or "fiducial" stations. In this observing scheme, the observations for the "master" station(s) are in common to most or all the other observing sessions for a project. The data for observing sessions linked by a master station can be processed simultaneously either in the fixed orbit or adjusted orbit mode. This is usually called a network base line solution.

Other procedures for processing the simultaneous observations include processing single or session base line solutions. In a session base line solution, all data collected simultaneously during an observing session are combined for simultaneous multiple-base line determinations.

Depending on the number of receivers available, project observing schemes that include one or more "master" stations may result in less efficient operations compared with the so-called "leapfrog" approach to planning the observing schemes. For example, efficiency is improved 20 to 35 percent when four receivers are operated in the "leapfrog" observing scheme rather than if one of the four receivers was used for continuous deployment at a "master" station.

On the other hand, the "master" station approach (also referred to as fiducial stations) might be highly desirable if the highest accuracy is required. For example, GPS observations might be collected continuously at the "master" stations located at sites of other space systems such as Very Long Baseline Interferometry (VLBI) or satellite laser ranging. These data can be processed while holding fixed the "master" station coordinates determined from the other space systems.

Table 4. -- Guidelines for GPS field survey procedures

Geometric relative positioning standards — Group / Order / ppm	AA / AA / 0.01	A / A / 0.1	B / B / 1.0	C / 1,2-I&II,3 / 10,20,50,100
Two frequency observations (1 and L2) required[a]: Daylight observations[b].....	Y	Y	Y	op
Recommended number of receivers observing simultaneously, not less than:·.·.........	5	5	4	3
Satellite Observations: RDOP values during observing session (meters/cycle)[d]....... [TO BE ADDED IN FUTURE VERSION]				
Period of observing session (observing span), not less than (min):				
[4 or more simultaneous satellite observations][e]				
Triple difference processing[f]...........	na	na	240	60-120
Other processing techniques[g]: General requirements[h][i].............	240	240	120	30-60
Continuous and simultaneous between all receivers, period not less than[i][j].	180	120	60	20-30
Data sampling rate - maximum time interval between observations (sec)...............	15	30	30	15-30
Minimum number of quadrants from which satellite signals are observed...........	4	4	3	3 or 2[k]
Maximum angle above horizon for obstructions[v] (degrees).................	10	15	20	20-40
Independent occupations per station(l):				
Three or more (percent of all stations, not less than)............................	80	40	20	10
Two or more (percent of stations, not less than):				
New stations............................	100	80	50	30
Vertical control stations..............	100	100	100	100
Horizontal control stations............	100	75	50	25
Two or more for each station of "station-pairs"(m)......................	Y	Y	Y	Y

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Table 4. -- Guidelines for GPS field survey procedures (continued)

Geometric relative positioning standards — Group / Order / ppm	AA / AA / 0.01	A / A / 0.1	B / B / 1.0	C / 1,2-I&II,3 / 10,20,50,100
Master or fiducial stations(n):				
Required, yes or no(o)......................	Y	Y	Y	op
If yes, minimum number......................	4	3	2	-
Repeat base line measurements, about equal number in N-S and E-W directions, minimum not less than (percent of total independently [nontrivial] determined base lines)..........	25	15	5	5
Loop closure, requirements when forming loops for post-analyses:				
Base lines from independent observing sessions, not less than..................	3	3	2	2
Base lines in each loop, total not more than.	6	8	10	10
Loop length, generally not more than (Km)....	2000	300	100	100
[NOTE: Also, see table 5]				
Loop closure (Continued):				
Base lines not meeting criteria for inclusion in any loop, not more than [percent of all independent nontrivial lines(p)]..........	0	5	20	30
Stations not meeting criteria for inclusion in any loop, not more than (percent of all stations)...............................	0	5	10	15
Direct connections are required: Between ANY adjacent (NGRS and/or new GPS) stations (new or old, GPS or non-GPS) located near or within project area, when spacing is less than (Km)..........................	30	10	5	3
Antenna setup:				
Number of antenna phase center height measurements per session, not less than...	3(q)	3(q)	2	2
Independent plumb point check required(r)....	Y	Y	Y	op

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Table 4. -- Guidelines for GPS field survey procedures (Continued)

Geometric relative positioning standards — Group / Order / ppm	AA / AA / 0.01	A / A / 0.1	B / B / 1.0	C / 1,2-I&II,3 / 10,20,50,100
Photograph (closeup) and/or pencil rubbing required for each mark occupied	Y	Y	Y	Y
Meteorological observations:				
Per observing session, not less than	3(s)	3(s)	2(t)	2(t) or op
Sampling rate (measurement interval), not more than (min)	30	30	60	60
Water vapor radiometer measurements required at selected stations?	op	op	N	N
Frequency standard warm-up time (hr)(u):				
Crystal	12	12	(u)	(u)
Atomic	1	1	(t)	(t)

LEGEND: nr - not required, na - not applicable, op - optional

REMARKS:

(a) If two-frequency observations can not be obtained, it is possible that an alternate method for estimating the ionospheric refraction correction would be acceptable, such as modeling the ionosphere using two-frequency data obtained from other sources.

Or, if observations are during darkness, single frequency observations may may be acceptable depending on the expected magnitude of the ionospheric refraction error.

(b) When spacing between any two stations occupied during an observing session is more than 50 km, two frequency observations may need to be considered for Accuracy Standards of Order 2 or higher.

(c) Multiple baseline processing techniques.

(d) Studies are underway to investigate the relationship of Geometric Dilution of Precision (GDOP) values to the accuracy of the base line determinations. Initial results of these studies indicate there is a possible correlation. It appears the best results may be achieved when the GDOP values are changing in value during the observing session.

(e) The number of satellites that are observed simultaneously cannot be less than the number specified for more than 25 percent of the specified period for each observing session.

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Table 4. -- Guidelines for GPS field survey procedures (Continued)

(f) Absolute minimum criteria is 100 percent of specified period.

(g) "Other" includes processing carrier phase data using single, double, nondifferencing, or other comparable precise relative positioning processing techniques.

(h) The times for the observing span are conservative estimates to ensure the data quantity and quality will give results that will meet the desired accuracy standard.

(i)

(j) Absolute minimum criteria for the data collection observing span is that period specified for an observing session that includes continuous and simultaneous observations. Continuous observations are data collected that do not have any breaks involving all satellites; occasional breaks for individual satellites caused by obstructions are acceptable, however, these must be minimized. A set of observations for each measurement epoch is considered simultaneous when it includes data from at least 75 percent of the receivers participating in the observing session.

(k) Satellites should pass through quadrants diagonally opposite of each other

(l) Two or more independent occupations for the stations of a network are specified to help detect instrument and operator errors. Operator errors include those caused by antenna centering and height offset blunders.

When a station is occupied during two or more sessions, back to back, the antenna/tripod will be reset and replumbed between sessions to meet the criteria for an independent occupation. To separate biases caused by receiver and/or antenna equipment problems from operator induced blunders, a calibration test may need to be performed.

(m) Redundant occupations are required when pairs of intervisible stations are established to meet azimuth requirements, when the distance between the station pair is less than 2 km, and when the order is 2 or higher.

(n) Master or fiducial stations are those that are continuously monitored during a sequence of sessions, perhaps for the complete project. These could be sites with permanently tracking equipment in operation where the data are available for use in processing with data collected with the mobile units.

(o) If simultaneous observations are to be processed in the session or network for base line determinations while adjusting one or more components of the orbit, then two or more master stations shall be established.

Table 4. -- Guidelines for GPS field survey procedures (Continued)

(p) For each observing session there are r-1 independent base lines where r is the number of receivers collecting data simultaneously during a session, e.g. if there were 10 sessions and 4 receivers used in each session, 30 independent base lines would be observed. (See appendix F and I.)

(q) A measurement will be made both in meters and feet, at the beginning, mid-point, and end of each station occupation.

(r) To ensure the antenna was centered accurately with the optical plummet over the reference point on the marker, when specified, a heavy weight plumb bob will be used to check that the plumb point is within specifications.

(s) Measurements of station pressure (in millibars), relative humidity, and air temperature (in°C) will be recorded at the beginning, midpoint, and end depending on the period of the observing session.

(t) Report only unusual weather conditions, such as major storm fronts passing over the sites during the data collection period. This report will include station pressure, relative humidity, and air temperature.

(u) The amount of warm-up time required is very instrument dependent. It is very important to follow the manufacturer's specifications.

(v) An obstruction is any object that would effectively block the signal arriving from the satellite. These include buildings, trees, fences, humans, vehicles, etc.

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One or more of the orbital parameters may be free in the adjustment while simultaneously solving for the base line vectors.

If a network solution is desired for ultimate accuracy, the observing scheme must include two or more "master" stations. The "master" stations should be located on opposite sides from the center of the project. All other criteria for the field procedures for either the "master" or other observing schemes are given in table 4. Which observing scheme is best, "leapfrog" or the "master" approach, will depend on the accuracy standard for the survey, the accuracy of the orbit coordinate data, and number of receivers available for the project. These and other factors will dictate the final observing strategy for the project.

For all surveys, the antenna must be stably located over the station mark for the duration of the observations within the allowable antenna setup error specified in appendix D. The height differences will be measured in feet and metric units and all will be recorded. Experience has demonstrated that blunders can be minimized by making this double measurement before and after each survey session.

The antenna phase center will be plumbed over the survey point using an optical plummet, collimator or similar instrument for control surveys. The each station occupation. The adjustment of optical plummets should be checked frequently, at least once per week or whenever there is an indication the plumb error exceeds the tolerance specified in appendix D. This check is for the purpose of determining gross plumb errors of 1 cm or more.

If an antenna is moved during an observing session, the set of observations for that session may not be acceptable. This will depend on such factors as the total data collection span before or after the antenna was moved, the quality of the data, and the quality and completeness of the data collected at the other observing stations.

The power source for the survey equipment should be stable and continuous especially for the high-accuracy surveys to minimize unnecessary breaks in the observations or damage to the equipment that would affect the quality of the data.

When observations of temperature and relative humidity are specified, these data shall be collected near the location of the antenna and at approximately the same height above the ground. Observations of wet-bulb and dry-bulb temperature readings should be recorded to the nearest 1.0°C. The relative humidity should be determined to the nearest 5 percent. Barometric readings at the station site should be recorded to the nearest millibar and corrected for any significant difference in height between the antenna phase center and location of the barometer. The meteorological instruments should be brought together and compared at least once per week and compared against a standard at least once per month. The logs shall include the name of manufacturer, model, and serial numbers of instruments used.

Office Procedures

Data Processing

Software to process the raw tracking data has been developed to handle either single or multiple base line input. The software incorporates a variety of models and differences in capabilities. Software adopted for processing the raw data must be certified as capable of producing results that meet the accuracy standards specified for a survey. Software can be certified by processing test data sets collected on FGCC 3-D test networks.

Numerous groups are investigating improvements to processing software. Major areas of work underway include: (a) orbit refinement modeling, (b) difference (single, double or triple) versus nondifference processing of carrier phase observations, (c) improved techniques for resolving carrier phase ambiguity and cycle-slips, and (d) improved atmospheric refraction modeling (ionosphere and troposphere).

All software must be able to produce from the raw data relative position coordinates and corresponding variance-covariance statistics which in turn can be used as input to three-dimensional network adjustment programs.

Criteria for processing and determining the quality of GPS relative positioning results are as follows (Remondi 1984 and Beutler et al. 1987):

1. The cutoff angle for data points should be no greater than 20.

2. The point position (absolute) coordinates for the station held fixed in each single, session, or network base line solution must be referenced to the datum for the satellite orbital coordinates (ephemerides). This datum is now called the World Geodetic System 1984 (WGS-84) (DMA 1987).

   The accuracy required for these coordinates will depend on the order of the survey. The order and corresponding accuracies are:

Order AA:	± 0.5 meter
Order A:	± 0.5 meter
Order B:	± 2.5 meter
Order 1 and lower:	± 25 meter

   In order of descending accuracies, the following are acceptable methods for estimating the fixed coordinates:

   a. Point position reduction of the GPS observations using Doppler smoothed pseudorange (code phase) measurements.

   b. Point position coordinates determined from unsmoothed GPS pseudorange measurements.

   c. Point position reduction of Transit Doppler observations using the precise ephemerides and transformed to WGS-84.

   d. Use of NAD 1983 published coordinates.

   e. Transformation of coordinates in a non-geocentric datum (e.g. NAD 1927) to the WGS-84 datum. In this method, the surveyor must be careful in obtaining transformation values that reflect with sufficient accuracy the differences between the non-geocentric local datum and the WGS-84 system.

3. Processing must account for the offset of antenna phase center relative to the station mark in both horizontal and vertical components.

4. As a rule of thumb, the number of simultaneous phase observations rejected (excluding those affected by cutoff angle and nonsimultaneous observations) for a solution should be less than 5 percent for accuracy standards AA, A and B, and 10 percent for the remaining standards.

5. Depending on the number of observations, quality of data, method of reduction, and length of base lines, the standard deviation of the range residuals in the base line solution should be between 0.1 and 2 cm for orders A, B, and 1; 1 to 4 cm for order 2; and, 1 to 8 cm for order 3.

6. The maximum allowable formal standard errors for the base line components may

depend on the particular software. With proper weighting in a fixed orbit solution, the values should be less than the expected accuracy for the orbit data. Typically, these range within 2 cm for base lines with lengths of less than 50 km.

Analysis and Adjustments

In practice, there will be two classifications for a GPS relative positioning survey. One would be based on the internal consistency of the GPS network adjusted independently of the local network control. This would be called the "geometric" classification. The second classification, if required, would be based on the results of a constrained adjustment where stations of the GPS survey network connected to the local network control are held fixed to vertical and horizontal coordinates in the National Geodetic Reference System (NGVD 1929 and NAD 1983). This is referred to as the "NGRS" classification.

Table 5 summarizes the specifications to aid in classifying the results for a GPS survey project.

Loop closures and differences in repeat base line measurements will be computed to check for blunders and to obtain initial estimates for the internal consistency of the GPS network.

Error of closure is the ratio of the length of the line representing the equivalent of the resultant errors in the base line vector components to the length of the perimeter of the figure constituting the survey loop analyzed. The error of closure is valid for orders A and B surveys only when there are three or more independently determined base lines (from three or more observing sessions) included in the loop closure analysis. For orders 1 and lower, independently determined base lines from a minimum of two observing sessions are required for a valid analysis. Loop closures incorporating only base lines determined from a common observing session (simultaneous observations) are not valid for analyzing the internal consistency of the GPS survey network.

After adjusting for any blunders, a minimally constrained (sometimes called a "free") least squares adjustment should be performed and the normalized residuals examined. The normalized residual is the residual multiplied by the square root of its weight, i.e. the ratio of the residual to the *a priori* standard error. Examining the normalized residuals helps to detect bad baseline vectors. In the "free" adjustment, one arbitrary station is held fixed in all three coordinates and the four bias unknowns (3 rotations and one scale parameter) are set to zero values (Vincenty 1987). The observation weights should be verified as realistic by inspecting the estimate of the variance of unit weight, which should be close to 1. However, in practice, it may be higher, perhaps in the range of 3 to 5 because for a particular GPS baseline solution software, the formal errors from the base line solutions may be too optimistic.

Vector component (relative position) standard errors computed by error propagation between points in a correctly weighted minimally constrained least squares adjustment will indicate the maximum achievable precision for the "geometric" classification.

Table 5. -- Office procedures for classifying GPS relative positioning networks independent of connections to existing control

Geometric relative positioning standards — Order: / ppm :	AA 0.01	A 0.1	B 1.0	1 10	2-I 20	2-II 50	3 100
<u>Ephemerides</u>:							
Orbit accuracy, minimum (ppm)............	0.008	0.05	0.5	5	10	25	50
Precise ephemerides required?............	Y[a]	Y[a]	Y	op	op	N	N
<u>Loop closure analyses(b)</u> - When forming loops, the following are minimum criteria:							
Base lines in loop from independent observations not less than............	4	3	2	2	2	2	2
Base lines in each loop, total not more than..................................	6	8	10	10	10	15	15
Loop length, not more than (Km)..........	2000	300	100	100	100	100	100
Base lines not meeting criteria for inclusion in any loop, not more than (percent of all independent lines)....	0	0	5	20	30	30	30
In any component (X,Y,Z), "maximum" misclosure not to exceed (cm).........	10	10	15	25	30	50	100
In any component (X,Y,Z), "maximum" misclosure, in terms of loop length, not to exceed (ppm)..................	0.2	0.2	1.25	12.5	25	60	125
In any component (X,Y,Z), "average" misclosure, in terms of loop length, not to exceed (ppm)..................	0.09	0.09	0.9	8	16	40	80
<u>Repeat base line differences</u>:							
Base line length, not more than (Km).....	2000	2000	500	250	250	100	50
In any component (X,Y,Z), "maximum" not to exceed (ppm)..................	0.01	0.1	1.0	10	20	50	100
<u>Minimally constrained adjustment analyses</u>:							
(Criteria is being developed and will appear in an updated version of this document)							

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Table 5. -- Office procedures for classifying GPS relative positioning networks independent of connections to existing control (continued)

REMARKS:
(a) The precise ephemerides is presently limited to an accuracy of about 1 ppm. By late 1989, it is expected the accuracy will improve to about 0.1 ppm. It is unlikely orbital coordinate accuracies of 0.01 ppm will be achieved in the near future. Thus to achieve precisions approaching 0.01 ppm, it will be necessary to collect data simultaneously with continuous trackers or fiducial stations. (see criteria for field procedures, table 5.) Then the all data is processed in a session or network solution mode where the initial orbital coordinates are adjusted while solvinmg for the base lines. In this method of processing the carrier phase data, the coordinates at the continuous trackers are held fixed.
(b) Between any combination of stations, it must be possible to form a loop through three or more stations which never passes through the same station more than once.

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The constrained least squares adjustment will use models which account for: the reference ellipsoid for the network control, the orientation and scale differences between the satellite and network control datums, geoid-ellipsoid relationships, the distortions and/or reliability in the network control, and instability in the control network due to horizontal and/or vertical deformation. A survey variance factor ratio will be computed to aid in determining the "NGRS" classification of the adjustment. The classification for the adjustment into the NGRS should not exceed the order for the combined control network.

The constrained adjustment determines the appropriate orientation and scale corrections to the GPS Baseline vectors so it with conform to the local network control. Because of possible significant inconsistencies in the network control between sections of the project area, it may be necessary to compute several sets of orientation and scale corrections. This is done by dividing the project area into smaller "bias groups", provided that in each such group there is sufficient existing control with adequate distribution that is tied to the GPS network (Vincenty 1987).

If reliable geoid height data are available, the adjustment to determine elevations should be done in terms of heights above the ellipsoid. However, useful estimates for elevations above mean sea level can be determined if geoidal height data are not available by fixing in an adjustment at least three stations with elevations. The stations with elevations must be well-distributed to permit fitting a plane through the three heights. The effect of ignoring the slope means that the geoidal slope is absorbed by two rotation angles (around the north and east axes in a horizon system) and geoidal heights are absorbed by the scale correction in a constarined 3-D adjustment (Vincenty). If there is one or more significant changes in the geoidal slope within the project area, the project can be divided into smaller "bias groups", provided there is at least three vertical control stations appropriately distributed within the "bias group" area.

The discussion related to "bias groups" points out the importance in the planning for a GPS survey project to insure there is included in the survey adequate connections to the horizontal and vertical control network.

See appendix G for examples of a network of points surveyed by GPS, each designed to meet different classification criteria. The field survey statistics are also summarized.

SUMMARY

Geometric relative positioning accuracy standards have been developed to meet classification requirements for control surveys and high-precision engineering surveys performed by GPS relative positioning techniques and other three-dimensional measurement systems such as VLBI. Relative positioning accuracies at the 1.0 cm + 1-2 ppm level can be achieved routinely from GPS carrier phase observations. The proposed standards augments the FGCC horizontal distance accuracy standards.

The specifications for geodetic surveys performed by GPS relative positioning techniques are based on extensive field and office experience gained at NGS, from special test surveys, and from reports prepared by numerous researchers within and outside of the United States. Much of the criteria reflects conservative estimates and will require further research and studies before they can be refined.

Development of the geometric accuracy standards and GPS relative positioning specifications is an evolutionary process that will continue for the foreseeable future or at least until after the Block II constellation of GPS satellites are deployed and fully operational in the early 1990's.

This document is presently undergoing a review by the U.S. Federal Geodetic Control Committee and will be considered for formal adoption. This process is expected to reach a conclusion by late summer 1988.

Until this document is formally adopted and published by the FGCC, users are cautioned to use this as only a guideline for the planning and execution of GPS relative positioning surveys.

REFERENCES

Beard, H., 1986: Memorandum on 3-D mark testing and results. National Geodetic Survey, April 4, 1986.

Beutler. G., Bauersima, I., Botten, S., Gurtner, W., Rothacher, M., and Schildknecht, T., 1987: Accuracy and biases in the geodetic positioning application of the Global Positioning System. Astronomical Institute, University of Berne, Berne, Switzerland, 18 pp.

DMA 1987: Department of Defense World Geodetic System 1984 - its definition and relationships with local geodetic systems. DMA Technical Report, 8350.2, Washington, DC, September, 125 pp.

DMA and NGS, 1986: Proceedings of the Fourth International Geodetic Symposium on Satellite Positioning, April 28-May 2, Applied Research Laboratories, The University of Texas at Austin, Austin, Texas.

Federal Geodetic Control Committee, 1984: Standards and Specifications for Geodetic Control Networks. National Geodetic Information Center, NOAA, Rockville, Md., 20852, September, 34 pp.

Goad, C.C. (Convener), 1985: Proceedings of the First International Symposium on Precise Positioning with Global Positioning System, April 15-19, Rockville, MD. National Geodetic Information Center, NOAA, Rockville, MD, 931 pp.

Greenwalt, C.R., and M.E. Shultz, 1962: Principles of error theory and cartographic applications. ACIC Technical Report No. 96, Aeronautical Chart and Information Center, St Louis, Missouri, February, 89 pp.

Hothem, L.D., and C.J. Fronczek, 1983: Report on test and demonstration of Macrometer model V-1000 interferometric surveyor. Federal Geodetic Control Committee, Report FGCC-IS-83-2, National Geodetic Information Center, NOAA, Rockville, MD, 36 pp.

Kaula, W, 1986: The need for vertical control. National Geodetic Information Center, NOAA, Rockville, MD, May, 31 pp. (preprint).

Kinlyside, D.A., 1988: A caparison of GPS baseline solutions when standard verses observed meterological values are used in the tropospheric model. Presented at New South Wales Staff Surveyors Conference, Australia, March, 13 pp.

McDonald, K., Parkinson, B, and McDonald, C.P., 1987: A survey of GPS user equipment, applications, and receiver technology trends. Proceedings of the The Institute of Navigation Satellite Division First Technical Meeting on GPS Navigation, Colorado Springs, Colorado, September 23-25.

Remondi, B.W., 1984: Using the global positioning system (GPS) phase observable for relative geodesy: modeling, processing, and results. Ph. D. dissertation, CSR-84-2, Center for Space Research, The University of Texas at Austin, Austin, TX, National Geodetic Information Center, NOAA, Rockville, MD, 360 pp.

Remondi, B.W., 1985: Global Positioning system carrier phase: description and use. Bulletin Geodesique, No. 59, pp. 361-377.

Sliwa, L., 1987: Some aspects of bench mark stability. Journal of American Congress on Surveying and Mapping, Vol. 47, No. 2, pp. 155-163.

Vincenty, T., 1987: On the use of GPS vectors in densification adjustments. Journal of American Congress on Surveying and Mapping, Vol. 47, No. 2, pp. 103-108.

Vonderohe, A.P., 1986: Positional accuracy standards, adjustments, and the multipurpose cadastre - some research issues. Journal of American Congress on Surveying and Mapping, Vol. 46, No. 2, pp. 131-135.

Zilkoski, D. and Hothem, L., 1988: GPS satellite surveys and vertical control. presented at ASCE Specialty Conference GPS '88 - Engineering Applications of GPS Satellite Surveying Technology, Nashville, Tennessee, May 11-14.

APPENDIX A.--FEDERAL GEODETIC CONTROL COMMITTEE MEMBERSHIP

The Federal Geodetic Control Committee (FGCC), chartered in 1968, assists and advises the Federal Coordinator for Geodetic Control and Related Surveys. The Federal Coordinator for Geodetic Control is responsible for coordinating, planning, and executing national geodetic control surveys and related survey activities of Federal agencies.

The Methodology Subcommittee of FGCC is responsible for revising and updating the Standards and Specifications for Geodetic Control Networks.

MEMBER ORGANIZATIONS

Department of Commerce
Department of Agriculture
Department of Defense
Corps of Engineers, U.S. Army
Department of Energy
Department of Housing and Urban Development
Department of Interior
Department of Transportation
National Aeronautics and Space Administration
Bureau of Land Management
International Boundary Commission

APPENDIX B.--ONE-DIMENSIONAL AND THREE-DIMENSIONAL (ELLIPSOIDAL AND SPHERICAL) ERRORS

Suppose the value m quantifies one of the components of the relative position between two marks, which may be, for example, relative height or the east-west base line component. Then the term "relative accuracy" for m will be defined as the ratio, ε/d, where the interval m- to m+ corresponds to the 95% confidence region for m while d equals the distance between two marks and ε equals the component error.

For a network of stations surveyed by GPS relative positioning techniques the three components of the relative position can be determined. The term "relative position accuracy" denotes the relative accuracy of the various components for a representative pair of network marks.

Consequently, a GPS network is said to have a relative positioning accuracy of 1 ppm (1:1,000,000) when each component of a representative base line has a relative accuracy of at least 1 ppm. The concept of relative position accuracy can be applied to networks where relative positions have been determined either by single-dimensional measurements or by three-dimensional space-based measurements (R. Snay, NGS, 1986 personal communications).

Accuracy standards for geometric relative positioning are based on the assumption that errors can be assumed to follow a normal distribution. Normal distribution applys only to independent random errors, assuming that systematic errors and blunders have been eliminated or reduced sufficiently to permit treatment as random errors.

Although, truly normal error distribution seldom occurs in a sample of observations, it is desirable to assume a normal distribution for ease of computation and understanding.

A three-dimensional error is the error in a quantity defined by three random variables. The components of a vector base line can be expressed in terms of dX, dY, and dZ. It is assumed that the spherical standard error (σ_s) is equal to the linear standard error for the components or $\sigma_s = \sigma_x = \sigma_y = \sigma_z$.

A one-sigma <u>spherical</u> standard error (σ_s) represents 19.9 percent probability. This compares to a one-sigma <u>linear</u> standard error (σ_x) which represents 68.3 percent probability.

At the 95 percent probability or confidence level, the spherical accuracy standard is $2.79\sigma_s$ compared to $1.96\sigma_x$ for a linear accuracy standard (Greenwalt and Shultz 1962).

The probability level of 95 percent is consistent with the <u>Standards and Specifications for Geodetic Control Networks</u> (FGCC 1984). On page 1-2 of this document, it is stated "... a safety factor of two ..." is "... incorporated in the standards and specifications." Since those accuracy standards were based on one-dimensional errors that exist in such positional data as elevation differences and observed lengths of lines, the factor of two, a $2\sigma_x$ linear accuracy standard, is a probability or confidence level of about 95 percent.

APPENDIX C.--CONVERSION OF MINIMUM GEOMETRIC ACCURACIES AT THE 95 PERCENT CONFIDENCE LEVEL FROM TABLE 1 TO MINIMUM "ONE-SIGMA" STANDARD ERRORS

The "one-sigma" three- and one-dimensional standard errors are computed by:

$\sigma_s = p/2.79$ and, $\sigma_x = p/1.96$

where, p = minimum geometric relative accuracies in (ppm) at the 95 percent confidence level

σ_s = "one-sigma" three-dimensional minimum error (ppm)

σ_x = "one-sigma" one-dimensional minimum error (ppm)

Tabulation of "one-sigma" errors for corresponding minimum geometric accuracies at the 95 percent confidence level.

Order	Class	Relative accuracies (95 percent) confidence level		Minimum geometric "One-sigma" standard errors			
				Three-dimensional (σ_s)		One-dimensional (σ_x)	
		p (ppm)	a (1:a)	(ppm)	(1:T)	(ppm)	(1:L)
AA	-	0.01	1:100,000,000	0.0036	1:279,000,000	0.005	1:200,000,000
A	-	0.1	1:10,000,000	0.036	1:27,900,000	0.05	1:20,000,000
B	-	1	1:1,000,000	0.36	1:2,790,000	0.5	1:2,000,000
1	-	10	1:100,000	3.58	1:279,000	5	1:200,000
2	I	20	1:50,000	7.17	1:140,000	10	1:100,000
2	II	50	1:20,000	17.9	1:56,000	25	1:40,000
3	I	100	1:10,000	35.8	1:28,000	50	1:20,000

APPENDIX D.--EXPECTED MINIMUM/MAXIMUM ANTENNA SETUP ERRORS

k = the repeatable setup error in (cm) for any component (horizontal and vertical) at the 95 percent confidence level

$$k = 0.1pd(\beta), \quad \text{where, } k_{min} = 0.3 \text{ cm and } k_{max} = 10 \text{ cm}$$

NOTE: The value for k_{min} is based on current estimates for expected setup errors when the antenna is set on a tripod at a total height of less than 5 m. When the antenna is set on a mast or twoer where the height is greater than 5 m, the esimated minimum value for k may be greater than 0.3 cm. On the other hand, if the antenna is mounted on a fixed or permanently installed stand, then K_{min} should be less than 0.1 cm.

The value for k_{max} is the expected largest value for the setup error; in practice, it should be much smaller than 10 cm, typically less than 1 cm.

p = minimum geometric accuracy standard in parts-per-million (ppm) (See table 1.)

d = distance between any two stations of a survey (km)

β = 0.05 = critical region factor for the 95 percent confidence level (1.00 - 0.95 = 0.05)

To convert setup error at the 95 percent confidence level to standard error (one-sigma), divide k by: 1.96 for 'linear' standard error, or 2.79 for 'spherical' standard error.

Tabulation of setup errors (k) in centimeters at 95 percent confidence level

Class		d = Distance between stations (km)										
	ppm	0.01	0.05	0.1	0.5	1	5	10	50	100	500	1000
AA	0.01	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
A	0.1	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
B	1	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.5	2.5	5
1	10	0.3	0.3	0.3	0.3	0.3	0.3	0.5	2.5	5	(10)	(10)
2-I	20	0.3	0.3	0.3	0.3	0.3	0.5	1.0	5	10	(10)	(10)
2-II	50	0.3	0.3	0.3	0.3	0.3	1.2	2.5	(10)	(10)	(10)	(10)
3-I	100	0.3	0.3	0.3	0.3	0.5	2.5	5	(10)	(10)	(10)	(10)

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APPENDIX E.--ELEVATION DIFFERENCE ACCURACY STANDARDS FOR GEOMETRIC RELATIVE POSITIONING TECHNIQUES

An elevation difference accuracy is the minimum allowable error at the 95 percent confidence level. For simplicity and ease of computations, elevation differences (dH) are assumed to be equal to orthometric height differences.

The height differences determined from space survey systems, such as GPS satellite surveying techniques, are with respect to a reference ellipsoid. These ellipsoid (geodetic) height differences (dh) can be converted to elevation differences (dh) by the relationship:

$$(dh) = (dH) - (dN)$$

where (dN) is the geoid height difference.

With accurate estimates for (dN) and adequate connections by GPS relative positioning techniques to network control points tied to National Geodetic Vertical Datum, elevations can be determine for stations with unknown or poorly known values.

> NOTE: If GPS ellipsoid height differences are being measured for the purpose of monitoring the change in height between stations, then it is not necessary to have any accurate information on the shape of the geoid. Thus, the accuracy of the height differences depends <u>only</u> on the accuracy of the GPS ellipsoid height differences.

The accuracy of the GPS derived elevations for points in a survey will depend on three factors: (1) accuracy of the GPS ellipsoid height differences, (2) accuracy of the elevations for the network control, and (3) accuracy of the geoid height difference estimates.

In the following table, elevation difference accuracy standards at the 95 percent confidence level are proposed. The order/class correspond to the proposed geometric relative position accuracy standards. At the high orders, the error is dominated by the accuracy for the (dN) values, whereas, for the lower orders, the major source of error is in the ellipsoid height differences.

> NOTE: In developing these standards, it is assumed that errors or inconsistencies in the vertical network control are negligible. Of course, this may not be true in many cases.

Elevation difference accuracy standards for geometric relative positioning techniques.

(95 percent confidence level)							
Order	Class	Minimum elevation difference accuracy standard		(From table 1) Minimum geometric relative position accuracy standard	Minimum geoid height difference accuracy standard	
		p_E (ppm)	1:e	p (ppm)	p_N (ppm)	1:n
AA	-	2	1:500,000	0.1	2	1:500,000
A	-	2	1:500,000	0.1	2	1:500,000
B	-	5	1:200,000	1	5	1:200,000
1	-	15	1: 67,000	10	10	1:100,000
2	I	20	1: 50,000	20	10	1:100,000
2	II	50	1: 20,000	50	20	1: 50,000
3	I	100	1: 10,000	100	40	1: 25,000

NOTE: **THESE ELEVATION DIFFERENCE ACCURACY STANDARDS ARE TO BE USED ONLY FOR ELEVATION DIFFERENCES DETERMINED INDIRECTLY FROM ELLIPSOID HEIGHT DIFFERENCE MEASUREMENTS.**

FOR DIRECT VERTICAL MEASUREMENT TECHNIQUES SUCH AS DIFFERENTIAL OR TRIGONOMETRIC LEVELING, USE ONLY THE ACCURACY STANDARDS GIVEN IN THE FGCC 1984 DOCUMENT, SECTION 2.2, PAGES 2-2 and 2-3.

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r = The number of GPS receivers used for each observing session

n = Minimum number of independent occupations per each station of a project

- If $n = 1$, (no check, no redundancy)
- If $n = 1.5$, (50 percent or more stations with 2 or more occupations)
- If $n = 1.75$, (75 percent or more stations with 2 or more occupations)
- If $n = 2$, (100 percent check, adequate redundancy)
- If $n = 3$, (excellent check, highest confidence)

NOTE: when, $r = 2$, n will always be 2 or greater.
when, $r > 2$, then $n = 1$, 2, 3, or more occupations.

m = Total stations for the project (existing and new)

s = Number of observing sessions scheduled for the project

d = Average number of observing sessions scheduled per observing day (e.g. 1 per day, 2 per day, 2.5 per day, etc.)

NOTE: Depends on required observing span, satellite availability, and transportation requirements.

x = Number of observing days, where $x = s/d$

y = Number of observing days scheduled per week, generally 5 to 7.

w = Number of workweeks, where $w = x/y = s/(d \cdot y)$

p = Production factor (based on historical evidence of reliability; ratio of proposed observing sessions for a project versus final number of observed sessions)

$$p = f/i,$$

where: f = final number of observing sessions required to complete the project

i = Proposed (initial) number of observing sessions scheduled for the project, where:

$$i = (m \cdot n)/r$$

FORMULAS:

$$s = (m \cdot n)/r + (m \cdot n)(p-1)/r + k \cdot m$$

where, k is a safety factor: $k = 0.1$ for local projects; within 100 km radius.
$k = 0.2$ for all other

x = estimated number of observing days for a project: $x = s/d$

w = estimated number of work-weeks for a project: $w = x/y$

v = estimated total vectors for a project: $v = r \cdot s(r-1)/2$

b = estimated independent vectors for a project: $b = (r-1)s$

EXAMPLE:

If	n =	1.75	independent occupations per station
	m =	50	total stations for project
	y =	5	observing days per week
	k =	0.2	safety factor
	r =	4	number of GPS receivers per observing session
	d =	2.5	average observing sessions per day
	p =	1.1	production factor
Then	s =	22 + 3 + 10 = 35 observing sessions	
	x =	14 observing days	
	w =	2.8 workweeks	
	b =	105 independent vectors	

COMMENTS:

In the equation to compute the number of observing sessions (s), if there were no sessions lost due to receiver malfunctions, and no additional sessions required to cover such factors as human error and irregular network configuration, then

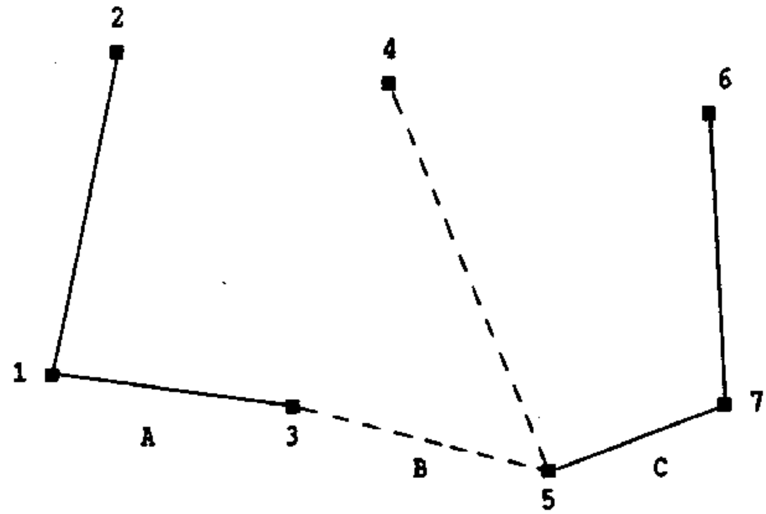
$$s = (m \cdot n)/r$$

However, the second part of the equation for computing "s" is to allow for additional sessions to offset scheduled sessions that may be lost due to equipment breakdown.

The third part of the equation, k(m), allows for additional sessions that may be required due to human error, irregular network configuration, etc.

APPENDIX G.--EXAMPLES OF GPS SURVEYS WITH SUMMARY OF STATISTICS USED TO CLASSIFY THE ORDER OF SURVEY BASED ON THE OBSERVING SCHEME AND DATA COLLECTION PROCEDURES

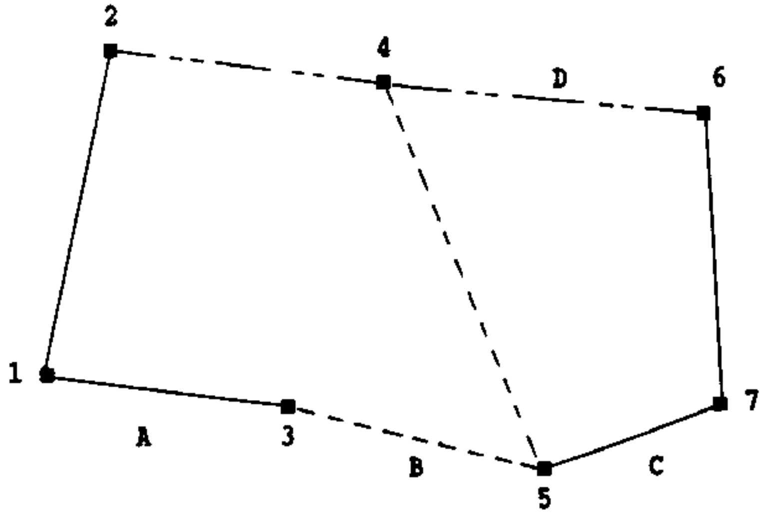
Example 1:



NOTE: Only nontrivial (independent) base lines are shown.

Observing sessions, total number (A,B, and C)	3
Receivers observing simultaneously	3
Stations, total number	7
Station occupations:	
Single occupations (no redundancy)	5
Two or more occupations, number/percent of all stations	2/29
Three or more occupations, number/percent of all stations	0/0
Base lines determined:	
All (trivial and nontrivial)	9
Independent (nontrivial)	6
Repeat base lines (N-S/E-W/percent of nontrivial)	0/0/0
Loop closure analyses:	
Valid loops formed?/Number of stations that can't be included	No/7
Loops containing base lines from (2 or)/(3 or more) sessions?	0/0
Geometric relative position classification (based on table 4)	None

Example 2:

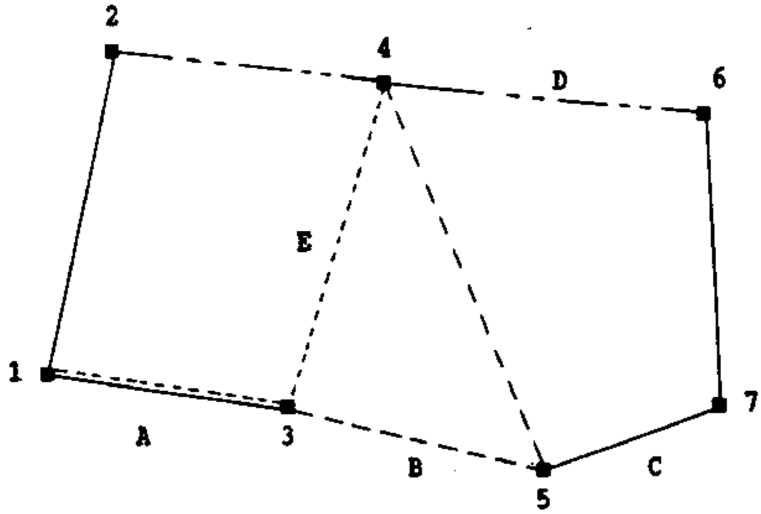


NOTE: Only nontrivial (independent) base lines are shown.

Observing sessions, total number (A,B,C, and D)	4
Receivers observing simultaneously.................................	3
Stations, total number...	7
Station occupations:	
Single occupations (no redundancy).................................	2
Two or more occupations, number/percent of all stations..........	5/71
Three or more occupations, number/percent of all stations........	0/0
Base lines determined:	
All (trivial and nontrivial).......................................	12
Independent (nontrivial)...	8
Repeat base lines (N-S/E-W/percent of nontrivial)..................	0/0/0
Loop closure analyses:	
Valid loops formed?/Number of stations that can't be included....	Yes[a]/0
Loops containing base lines from (2 or)/(3 or more) sessions?....	0/2
Geometric relative position classification (based on table 4)......	Order "2-II"

(a) Loops formed: 1- 1(A)3 + 3(B)5 + 5(B)4 + 4(D)2 + 2(D)1 Includes 3 sessions
2- 5(C)7 + 7(C)6 + 6(D)4 + 4(B)5 Includes 3 sessions

Example 3:



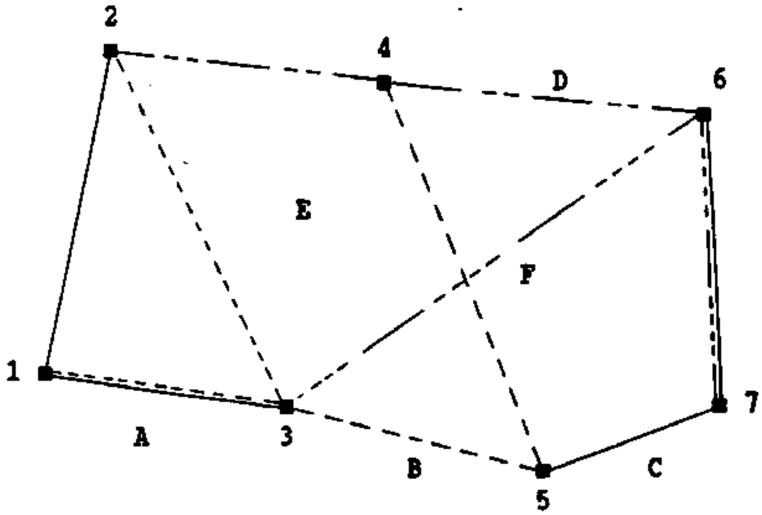
NOTE: Only nontrivial (independent) base lines are shown.

Observing sessions, total number (A,B,C,D, and E)..................	5
Receivers observing simultaneously.................................	3
Stations, total number...	7
Station occupations:	
Single occupations (no redundancy).................................	1
Two or more occupations, number/percent of all stations..........	6/86
Three or more occupations, number/percent of all stations........	2/29
Base lines determined:	
All (trivial and nontrivial).......................................	15
Independent (nontrivial)...	10
Repeat base lines (N-S/E-W/percent of nontrivial)..................	0/1/10
Loop closure analyses:	
Valid loops formed?/Number of stations that can't be included....	Yes[a]/0
Loops containing base lines from (2 or)/(3 or more) sessions?....	1/2
Geometric relative position classification (based on table 4)......	Order "1"

(a) Loops formed:

1-	1(A)3 + 3(E)4 + 4(D)2 + 4(A)1	Includes 3 sessions
2-	3(B)5 + 5(B)4 + 4(E)3	Includes 2 sessions
3-	5(C)7 + 7(C)6 + 6(D)4 + 4(B)5	Includes 3 sessions

Example 4:



NOTE: Only nontrivial (independent) base lines are shown.

Observing sessions, total number (A,B,C,D,E, and F)	6
Receivers observing simultaneously................................	3
Stations, total number..	7
Station occupations:	
Single occupations (no redundancy)...............................	0
Two or more occupations, number/percent of all stations..........	7/100
Three or more occupations, number/percent of all stations........	3/43
Base lines determined:	
All (trivial and nontrivial).....................................	18
Independent (nontrivial)...	12
Repeat base lines (N-S/E-W/percent of nontrivial).................	2/1/25
Loop closure analyses:	
Valid loops formed?/Number of stations that can't be included....	Yes[a]/0
Loops containing base lines from 2 or / 3 or more sessions?......	0/4
Geometric relative position classification (based on table 4)......	Order "B"

NOTE: If one additional session was observed where session G would include stations 1,2 and 5 (or 7), then the survey would be classified with an Order of "A".

(a) Loops formed:	1-	1(A)3 + 3(E)4 + 4(D)2 + 2(A)1	Includes 3 sessions
	2-	3(B)5 + 5(C)7 + 7(C)6 + 6(F)3	Includes 3 sessions
	3-	6(D)4 + 4(B)5 + 5(C)7 + 7(F)6	Includes 3 sessions
	4-	1(E)3 + 3(B)5 + 5(B)4 + 4(E)2 + 2(A)1	Includes 4 sessions

APPENDIX H.--SPECIFICATIONS AND SETTING PROCEDURES FOR THREE-DIMENSIONAL MONUMENTATION

May 11, 1988

A. Materials required for each marker:

1. Rod, stainless steel, 4-foot sections
2. Rod, stainless steel, one 4-5 inch
2. Studs, stainless steel, 3/8 inch
3. Datum point, stainless steel, 3/8 inch bolt
4. Spiral (fluted) rod entry point, standard
5. NGS logo caps, standard, aluminum
6. Pipe, schedule 40 PVC, 5 inches inside diameter, 2-foot length
7. Pipe, schedule 40 PVC, 1 inch inside diameter, 3-foot length
8. Caps, schedule 40 PVC, (Slip-on caps centered and drilled to 0.567 inch ±0.002)
9. Cement for making concrete
10. Cement, PVC solvent
11. Loctite (2 oz. bottle)
12. Grease
13. Sand (washed or play)

B. Setting procedures:

1. The time required to set an average mark using the following procedures is 1 to 2 hours.

2. Using the solvent cement formulated specifically for PVC, glue the aluminum logo cap to a 2-foot section of 5-inch PVC pipe. This will allow the glue to set while continuing with the following setting procedures.

3. Glue the PVC cap with a drill hole on one end of a 3-foot section of schedule 40 PVC pipe 1-inch inside diameter. Pump the PVC pipe full of grease. Thoroughly clean the open end of the pipe with a solvent which will remove the grease. Then glue another cap with drill hole on the remaining open end. Set aside while continuing with the next step.

4. Using a power auger or post hole digger, drill or dig a hole in the ground 12-14 inches in diameter and 3 1/2 feet deep.

5. Attach a standard spiral (fluted) rod entry point to one end of a 4-foot section of stainless steel rod with the standard 3/8 inch stud. On the opposite end screw on a short 4 to 5 inch piece of rod which will be used as the impact point for driving the rod. Drive this section of rod with a reciprocating driver such as Whacker model BHB 25, Pionjar model 120, or another machine with an equivalent driving force.

6. Remove the short piece of rod used for driving and screw in a new stud. Attach another 4-foot section of rod. Tighten securely. Reattach the short piece of rod and drive the new section into the ground.

B. Setting procedures (continued):

7. Repeat step 6 until the rod refuses to drive further or until a driving rate of 60 seconds per foot is achieved. The top of the rod should terminate about 3 inches below the ground surface.

8. When the desired depth of the rod is reached, cut off the top removing the tapped and threaded portion of the rod leaving the top about three inches below ground surface. The top of the rod then must be shaped to a smooth rounded (hemispherical) top, using a portable grinding machine to produce a datum point. The datum point must then be center punched to provide a plumbing (centering) point.

   NOTE: For personnel that may not have the proper cutting or grinding equipment to produce the datum point, the following alternative procedure should be used if absolutely necessary. When the desired depth of the rod is obtained (an even 4-foot section), thoroughly clean the thread with a solvent to remove any possible remains of grease or oil that may have been used when the rod was tapped. Coat the threads of the datum point with Loctite and screw the datum point into the rod. Tighten the point firmly with vise grips to make sure it is secure. The datum point is a stainless steel 3/8 inch bolt with the head precisely machined to 9/16 inch.

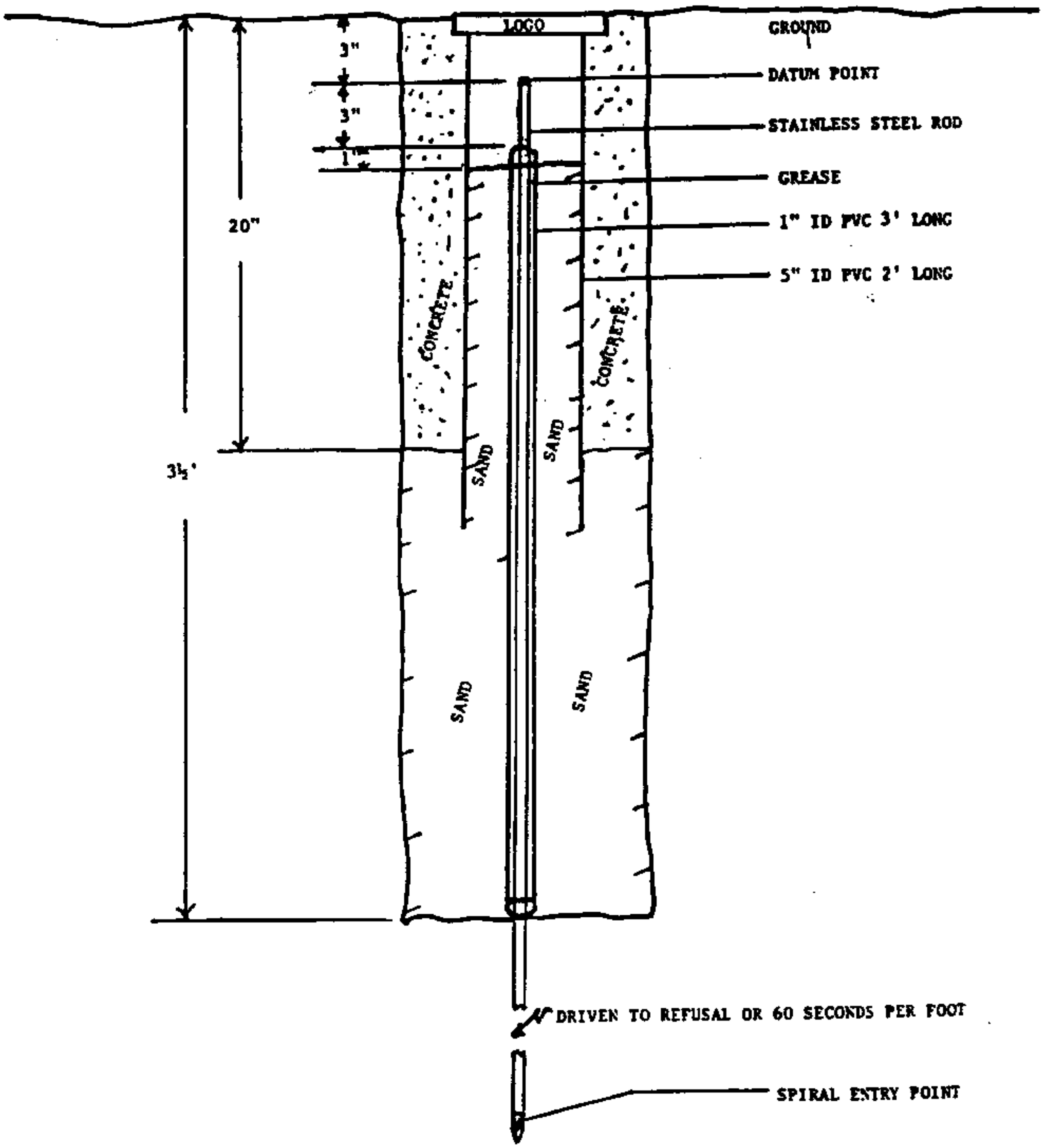
9. Insert the grease filled 3-foot section of 1-inch PVC pipe (sleeve) over the rod. The rod and datum point should protrude through the sleeve about 3 inches.

10. Backfill and pack with sand around the outside of the sleeve to 20 inches below ground surface. Place the 5-inch PVC and logo cap over and around the 1-inch sleeve and rod. The access cover on the logo cap should be flush with the ground. The datum point should be about 3 inches below the cover of the logo cap.

11. Place concrete around the outside of the 5-inch PVC and logo cap, up to the top of the logo cover. Trowel the concrete until a smooth neat finish is produced.

12. Continue to backfill and pack with sand inside the 5-inch PVC and around the outside of the 1-inch sleeve and rod to about 1 inch below the top of the sleeve.

13. Remove all debris and excess dirt to leave the area in the condition it was found. Make sure all excess grease is removed and the datum point is clean.



Schematic of the NGS 3-D marker