California Geodetic Control Committee

Mr. Gregory A. Helmer, PLS Chairperson
c/o Robert Bein, William Frost & Associates
PO Box 57057 Irvine, California 92619-7057

SPECIFICATIONS FOR GEODETIC CONTROL NETWORKS USING HIGH-PRODUCTION GPS SURVEYING TECHNIQUES

Version 2.0, July 1995

Prepared by:

Michael Anderson, PLS
Don D'Onofrio
Gregory A. Helmer, PLS
Wayne Wheeler, Jr., PLS

Specifications for Geodetic Control Networks Using
High-Production GPS Surveying Techniques
Version 2.0, July 1995

TABLE OF CONTENTS

ABSTRACT

Technological advancements in GPS data collection and processing have provided the ability to perform precise geodetic control surveying in a fraction of the time previously required by classical static GPS methods. Experience with procedures such as kinematic, repeat occupation, and fast ambiguity resolution has shown that results equivalent to the existing geodetic accuracy standards (FGCC, 1989) can be achieved. This ability, together with the proliferation of continuously operating reference stations (CORS) and the expanding use of geodetic control in support of spatial information systems, demands the adoption of reliable procedures to support high-production GPS surveying activities. The continuing evolution of GPS hardware and software complicates the formulation of certain details regarding specifications for geodetic control surveys, but there is now fairly broad consensus within the GPS community regarding proper methodology.

Revision 2.0 of this document incorporates the comments and practical experiences of GPS users over the two-years since its first publication. More significantly, the definition of accuracy has been revised to adopt spatial accuracy rather than relative precision. Classification of accuracy is defined separately for horizontal and vertical, and is further subdivided by local accuracy (the confidence in a given point's coordinates relative to the coordinates of the other points in the network) and network accuracy (the confidence in a given point's coordinates with respect to the geodetic datum).

The collective experiences of this committee, together with the published experiments of others, as well as discussions with individuals throughout the GPS industry, serve as the basis for these specifications. The specifications are designed to supplement the FGCS proposed spatial accuracy standards and existing relative GPS standards and specifications, so as to provide for controlled utilization of high-production GPS surveying methods for precise geodetic control projects. As such, they are guidelines to be used in the collection and processing of GPS data for geodetic control purposes. While it may be possible to obtain a given accuracy without adherence to all of these requirements, each has been carefully crafted to provide a specific level of confidence. The burden of proof rests with the practioner as to why a requirement should be waived and to the validity of the accuracy claim.

INTRODUCTION

The impact of GPS on geodetic control surveying, since its introduction in the 1980's, is nothing short of astounding. GPS has put into the hands of the surveying community a powerful tool for the establishment of precise geodetic control. Just as electronic distance measurement and desk top computers profoundly influenced the practice of land surveying, GPS has contributed to the profession's evolution. This evolution has brought with it reduction in cost and an increased demand for geodetic control services.

It is reasonable to expect that changes in technology and the marketplace experienced during this previous period will continue into the future. Utilization of high-production GPS surveying techniques such as kinematic, repeat occupation, fast ambiguity resolution and ambiguity-resolution-on-the-fly (AROF) are motivated by the demands of the market place to provide more and better service from fewer dollars. This motivator is as forceful in a public sector which is meeting increased demands to downsize as it is to a profit-driven private sector. Experience has shown that current hardware and software has the capability to achieve geodetic-quality accuracy levels under conditions and restrictions quite different from those defined in the current GPS specifications (Remondi, 1986) (Frei and Beutler, 1990). Failure to address these techniques and their proper application would be remiss, if we want to continue the successful use of GPS surveying.

Consumers of geodetic control services, as well as practitioners, have a very real need to rely upon practical standards and specifications which will ensure a given level of product consistency. The existing documents addressing geodetic control standards and specifications; *Standards and Specifications for Geodetic Control Networks* (FGCC, 1984) and *Geometric Geodetic Accuracy Standards and Specifications for Using GPS Relative Positioning Techniques* (FGCC, 1989), have provided leadership in guiding the establishment of precise geodetic control. Without this leadership it is questionable that GPS could have achieved its current level of acceptance.

Four items are considered paramount for ensuring the success of a geodetic control project. Regardless of how the observations were obtained, the completed network must provide the following:

- Elimination or reduction of known and potential systematic error sources.

- Sufficient redundancy to clearly demonstrate the stated accuracy.

- Adequate analysis and data processing.

- Sufficient documentation to allow verification of the results.

The specifications listed herein are directed at these concerns. These specifications are intended to be used as quality control for GPS geodetic control surveys utilizing techniques other than "static" methods. They are meant to augment, rather than replace the existing document addressing GPS static survey methodology (FGCC, 1989). The proposed spatial accuracy standards (FGCS, 1994) are endorsed herein and briefly described. These high-production GPS surveying specifications do not address accuracies higher than Band IV (< 0.005 meter). While higher accuracy has been demonstrated using similar techniques (Bock, 1991), experience with high-production GPS surveying at this accuracy level is very limited, and the need for this type of control is not influenced by the same demands which have prompted these specifications.

To state that a survey has been conducted to this document's standards, three groups of criteria must be satisfied:

- Adherence to the specifications for methodology, data collection, and processing.

- Achievement of accuracy standards by the survey's results.

- Preparation and archiving of documentation showing compliance with these specifications and standards.

Whereas the documentation requirements are the same for any survey, certain aspects of the specifications vary depending on the survey's methodology and intended order of accuracy.

Finally, as the resources available to the profession progress, its knowledge base must also progress. Clearly it is not possible to legislate quality work. Standards and specifications are easily defeated by the ill-prepared or unprofessional. Education gained through academics, personal experience, and the open exchange of ideas and experiences of the profession remains the best method for ensuring the proliferation of reliable geodetic control. It is hoped that these specifications will contribute to this process.

DEFINITIONS

STATIC GPS

Static GPS surveying is the traditional method for relative positioning. An extended observation period (typically 30 to 120 minutes) through a change in satellite geometry is recorded on one or both carrier frequencies. Typical data processing involves triple differencing to determine initial baseline components. Triple difference baseline estimates are used to identify cycle slips and clean the data for double difference processing. Double differencing can be performed individually by station pairs or in a simultaneous solution for each session. Various mathematical models accounting for satellite orbits, ionospheric refraction, and carrier phase ambiguities can be held fixed or allowed to float in the solution. The strongest solutions have successfully resolved the carrier phase ambiguities and held these values fixed for the final baseline determination.

INTERMITTENT KINEMATIC GPS

Sometimes known as stop-and-go kinematic, intermittent kinematic GPS, relies upon prior knowledge of the carrier phase ambiguities to successively resolve baselines with a very minimal amount of data. The double difference algorithm is employed holding fixed the known integer values for each successive solution. As little as a single epoch is all that is necessary provided integer values can be maintained by continuously tracking a minimum of four satellites as the receiver(s) are moved from point to point. Integer values can be computed from observations made on a known baseline, performing an antenna swap, or by using on-the-fly ambiguity resolution.

REPEAT OCCUPATION GPS

Repeat occupation, also know as pseudo static or pseudo kinematic uses two or more observations of the same baseline to compute integer terms from the change in satellite geometry *occurring over the time period between observations. The receiver pairs occupy points for relatively short (5 to 10 minute) session and then return to the same points for a second or third session after an hour or more has elapsed. These multiple observations are combined in the double difference processing where the change in satellite geometry is exploited to resolve the integer terms.*

FAST AMBIGUITY RESOLUTION

Fast ambiguity resolution has also been referred to as fast static, rapid static, and quick static. The technique involves utilization of multiple observables (e.g. L_1 and L_2 carriers, C/A, P and Y codes), and various combinations thereof, to provide the limits for a search space of possible integer combinations. The processing software tests these possible integer values to identify the most probable candidates. Integer resolution is most successful with data collected during periods of five or more visible satellites and with all observables recorded. However, some studies have indicated acceptable results with fewer observables and with observation periods as short as one minute. Significant of note is the fact that the GPS receivers do not necessarily need to remain static during the ambiguity resolution process. So called ambiguity resolution on-the-fly (AROF) combined with kinematic methods could potentially become the norm for a great many applications.

DIFFERENTIAL GPS

Differential GPS has not often been associated with precise geodetic control work. Research and development is being conducted, however, which could result in the viability of this technique, or some hybrid, for many applications including perhaps, geodetic control surveying. Differential GPS positioning does not attempt to solve the relative position between stations so much as it attempts to resolve the inherent errors in a single autonomous position. This is typically done by making observations at one or more known stations while code-phase measurements (pseudo ranges) are recorded at the remote location. The position of the known station is used to compute error values for the pseudo ranges which are applied to the remote data.

HORIZONTAL AND VERTICAL ACCURACY STANDARDS

As previously stated, these specifications accept the spatial accuracy standards proposed by the Federal Geodetic Control Subcommittee (FGCS, 1994). Thirteen Bands are defined by the radius of the relative positional error circle of 95% confidence. Table 1 lists the accuracy Bands from the FGCS proposal. Horizontal, ellipsoidal height, and orthometric height classifications retain the same nomenclature although each is classified independently. Similar to FGCC, 1989, two types of accuracy classification are defined. For the network accuracy measure, the relation is between the control point in question, and the nearest COR station supported by NGS. For the local accuracy measure, the relative accuracies are computed along the measured lines used to establish the control point in question. These accuracies are computed by random error propagation from a least squares adjustment, where the survey measurements have the correct weights, and where constraining datum values are weighted using one-sigma network accuracies of the existing network control. Details regarding computation of the radius of the 95% confidence relative error circle can be found in the FGCS proposal. The following material, taken from the FGCS proposal, is intended to familiarize the reader with the concepts of spatial accuracy classification. For a more complete description the reader must consult the referenced document or its successor.

These specifications address procedures for achieving Bands IV through VIII (i.e. 0.005 to 0.500 meters), the upper and lower margins of which reach the practical limitations for which these procedures were designed. Bands 0 through III, and IX through XII are considered to be outside of the scope of these specifications and therefore not considered herein.

Table 1 **Spatial Accuracy Standards**

Horizontal, Ellipsoidal Height, and Orthometric Height Classifications

Classification Band	95% Confidence		
Band 0	reserved for CORS		Outside the scope of these specifications
Band I	<= 0.001	meter	
Band II	0.001 - 0.002	meter	
Band III	0.002 - 0.005	meter	
Band IV	0.005 - 0.010	meter	Horizontal and vertical accuracy bands included in these specifications
Band V	0.010 - 0.020	meter	
Band VI	0.020 - 0.050	meter	
Band VII	0.050 - 0.100	meter	
Band VIII	0.100 - 0.500	meter	
Band IX	0.500 - 2.000	meter	Outside the scope of these specifications
Band X	2.000 - 5.000	meter	
Band XI	5.000 - 10.000	meter	
Band XII	> 10.000	meter	

As a test for systematic error, comparison is made between coordinates from a minimally-constrained least squares adjustment of the survey data and the NSRS coordinates at established control points. Horizontal position shifts are computed at these common points, i:

$$dS_i = (dN^2 + dE^2)^{1/2}$$

where

dN	=	apparent position shift in latitude
dE	=	apparent position shift in longitude

Also vertical shifts (ellipsoidal and othometric) are computed by:

dV = apparent height shift - ellipsoid or orthometric height

The horizontal and vertical position shifts are tested against a 95% confidence value. This confidence value is a combination of three error terms the 95% network accuracy of the fixed control point in the survey adjustment, the radius of a 95% error circle at point i relative to the fixed control point, and the 95% network accuracy of the NSRS position at point i. The 95% position tolerance, T_{95}, at point i is computed:

$$T_{95} = (N_{fix}^2 + r_i^2 + N_i^2)^{1/2} \quad \text{for horizontal component}$$
$$T_{95} = (N_{fix}^2 + V_i^2 + N_i^2)^{1/2} \quad \text{for vertical component}$$

Where

N_{fix}	=	fixed position network accuracy, 95%
r_i	=	survey adjustment error circle radius, 95%
V_i	=	survey adjustment height difference error, 95%
N_i	=	NSRS position network accuracy, 95%

The network accuracies, N_{fix} and N_i, are obtained from published values of the NGS database. The error circle radii, r_i, and height difference errors, $V_{i,}$ are computed from the minimally-constrained adjustment of the survey data. The 95% position tolerances, T_{95}, are not used to assign the accuracy classification. They are used to check the position shifts dS_i and height shifts dV_i.

The position shifts should seldom exceed the combined 95% tolerance. If this is not the case, or if common trends are obvious, then both the survey and network measurements must be scrutinized for sources of the problem. Possible sources of systematic error are crustal motion, datum distortion, uncalibrated survey equipment, and improper data reduction procedures.

Accuracy classification of a GPS relative positioning survey may be determined in one of two ways, dependent upon the project application. A network classification may be made where the confidence tests are made with respect to the geodetic datum (i.e. COR stations). This type of classification is applicable to projects which are to be included in the National Spatial Reference System (NSRS) or the California Spatial Reference System (CSRS). A local accuracy classification may be made to classify the relative confidence of new project coordinates with respect to the coordinates of other points in the particular project. Local accuracy classification is applicable to project-specific control systems where the resulting coordinates are not intended to be used for the control of future geodetic control networks.

NETWORK DESIGN

Network design, for the purposes of the this document, includes the determination of the number and location of existing control stations for network constraints, the selection of new project station locations, and the relative dispersion of network observations. The subject of observational redundancy, itself contributory to network design, will be discussed under "DATA COLLECTION".

Space-based measurement systems, such as GPS, are not significantly affected by such factors as network shape or intervisibility (JPO, 1991) (Leick, 1992). This provides the opportunity to focus upon the intent of the project control, and geographically sensitive issues such as geoid modeling, rather than the limitations of the measurement technique. However, network design does have relevance both for the elimination or reduction of potential error sources as well as for providing adequate ties to the existing geodetic reference system. These concerns may be addressed by the choice of which existing control stations should be included, as well as the planning of new station locations and network observation periods. GPS-derived orthometric heights are particularly sensitive to the distribution of observations and network constraints.

To meet a network or local accuracy classification, a GPS project must be connected to sufficiently accurate and well distributed existing control. Network constraints of a lesser accuracy than that intended for the project may be included; however, additional control of a higher accuracy will also be necessary due to limitations imposed by the computation of the 95% error circle. All of the control stations to which the network will be constrained must have positions known on the same datum and epoch since additional uncertainty in the time-dependent positioning models (TDP), which would need to be added into the network constraints, will likely add unacceptable levels of uncertainty to the network (Snaye, 1993). Control stations of the CSRS should be given preference; however, certain special projects may have legitimate need for other geodetic references. The minimum number of horizontal and ellipsoidal network constraints is stated herein, with their location being distributed in different quadrants relative to the center of the project. Where existing horizontal and/or ellipsoidal control on a common datum and epoch is available, all such stations lying within two kilometers of the survey's boundaries must be included in the survey, up to a maximum requirement of 10% of the total network stations. Requirements for orthometric height constraints are dependent upon geoid slope, project extent, desired accuracy, and the density of the gravity database. These issues are addressed under "GEOID MODELING" herein.

Independent observations are the only measurement considered by these specifications.

Trivial vectors from multiple-GPS-receiver sessions should not be included in the network; this is to prohibit adjusting the same observations more than once and misstating the network degrees of freedom. For any given multiple-receiver session there are [n(n-1)]/2 total independent and trivial vectors possible, where n = the number of GPS receivers observing simultaneously. The number of independent vectors is n-1. Baseline solutions may be omitted from the final least squares adjustment only if the remaining network meets the minimum requirements listed under "DATA COLLECTION". For a station to qualify for an accuracy classification, network or local, it must meet the herein-listed accuracy standards relative to all other stations in the network and/or datum, whether or not there was a direct connection between them.

The following, Table 2 , lists additional specifications for network design:

Table 2 **Network Design**

Spatial Accuracy Classification	HORIZONTAL COMPONENT			ELLIPSOID HEIGHT	
	0.005-0.020 meters	0.020-0.050 meters	0.050-0.500 meters	0.005-0.050 meters	0.050-0.500 meters
Network Control:					
Minimum number of stations and quadrants:	3	3	2	4	2
Maximum distance between project's outer boundary and network control stations:	50 km.	50 km.	50 km.	20 km.	20 km.
Initial Position:					
Maximum 3-d error for the NAD83 coordinates input for the initial station in any baseline solution:	10 m.	20 m.	50 m.	10 m.	20 m.
Baseline Connections:					
A baseline observation must be made between any two stations (1and 2), where their spacing is less than (___)% of the otherwise shortest direct connection to either station.	10%	N/A	N/A	30%	20%

DATA COLLECTION

Redundancy can provide proof of the precision to which a measurement is made. In order for this proof to be meaningful, the inclusion of possible error sources must not be systematically duplicated in the redundant measurements. A well-understood example from terrestrial surveying is that, in an optically-read theodolite angle measurement, if all repeat angles were turned with the same horizontal circle reference set on the backsight, inaccuracy in that particular portion of the theodolite's circle would not be made apparent. Redundancy in a GPS survey is achieved primarily by way of a change in the relative geometry of the satellite constellation. This is easily visualized when one compares the dominance of the satellite geometry at a distance of 20,000 km. versus the insignificant separation between typical project stations. For GPS surveys, the geometry of the satellite constellation must be different for repeat station observations in order to eliminate potential sources for systematic errors do to multipath, orbit bias, and unmodeled ionospheric and tropospheric delay. Even if the repeat station observation is made on another day, data must be collected at a different sidereal time in order to obtain a different satellite configuration. The minimum times between repeat station observations are listed in "Data Collection" Table 3. Redundant observations also provide the additional verification of centering errors and a second set of antenna height measurements.

Two unique situations regarding redundant GPS observations deserve comment. First, with the abundance of satellites visible at any given time and the availability of all-in-view GPS receivers, the possibility exists to compute quasi-independent vectors from the same observation session. A quasi-independent vector would use a different sub-group of four or more satellites during the same occupation. This technique is not considered valid fulfillment of the repeat station observation or repeat baseline requirements. Second, the pseudo-static technique by definition necessitates repeat station occupations. This paired observation is defined as one station observation for the purposes of these specifications. However, three station occupations could provide two independent baseline solutions, providing that the above-stated minimum sidereal time difference separates the first and third observations. For all other types of GPS surveying methodology, each station occupation can provide one station observation.

Each piece of equipment used during the survey to center the antenna over the stations (tribrach, optical plummets, fixed-height rod, etc.) must be periodically tested, and if necessary, adjusted. These tests may either be conducted by a survey equipment repair/maintenance facility or by the surveyor performing the survey. In either case, the testing procedure must provide results consistent with the maximum allowable setup error defined herein, and documentation for these tests must be included with the survey report. The minimum test schedules are:

1. For accuracy bands less than 0.020 meters, each centering devise must be tested within 30 days prior to a survey's commencement and within 10 days after a survey's completion. For long-term projects, testing must occur at regular 30-day intervals.

2. For accuracy bands greater than 0.020 meters, each centering devise must be tested within 6 months prior to a survey's commencement and within 6 months after a survey's completion.

Observation errors (weights) used in the least squares adjustment must include estimates of the total antenna centering error consisting of: equipment precision as confirmed by said testing, antenna height measurement error, and phase-center stability. Phase center drift has been shown to be as much as 1.5 cm horizontally and 4 cm vertically (Braum, Rocken & Johnson, 1994) and is different between L_1 and L_2 frequencies. Where mixed antenna types are used on a project, phase center modeling must be employed to acheive horizontal accuracies better than 1 cm or vertical accuracy better than 2 cm.

Each station occupation is to be an independent setup and data collection process. Antenna heights must be measured, both in feet and in meters, at the beginning and the end of each occupation. If the measurement is made to the antenna's ground plane, rather than to a manufacture-determined "mark", then either the beginning or ending height measurement must be made to at least three equidistant points along the ground plane's edge. The use of fixed-height setup equipment is recommended for all precise control applications, particularly for vertical control projects. Clear photographs or legible rubbings of the station monument should be collected for each station occupation. Notes must be prepared for each occupation and must include as a minimum:

1. The beginning and ending times for the occupation.
2. The name of the surveyor performing the occupation.
3. The receiver identifier (make model, serial number)
4. The centering devise identifier.
5. All antenna height measurements (in meters and feet).
6. The station identifier (name and/or survey point number)
7. A description of the monument and center mark.
8. Monument rubbing/photograph if required.

The duration of any GPS observation session is a greatly variable quantity depending upon, among other things, the desired level of accuracy, satellite geometry, the observables recorded, the observation techniques, and the processing software. Suffice it to say that adequate results have been demonstrated with observation times of less than a minute to several hours or days. Sufficient redundancy teamed with proper analysis and documentation will mitigate concerns over observation duration. Actual errors will be reflected in the network adjustment if a proper weighting strategy is employed with sufficient redundancy. The following, Table 3, lists additional specifications for planning and data collection.

Spatial Accuracy Classification	HORIZONTAL COMPONENT			ELLIPSOID HEIGHT	
	0.005-0.020 meters	0.020-0.050 meters	0.050-0.500 meters	0.005-0.050 meters	0.050-0.500 meters
Repeat station observation percent of number of stations:					
Two times:	100%	100%	80%	100%	100%
Three or more times:	10%	10%	0%	50%	25%
Sidereal time displacement between occupations (start time to next start)	60 min.	45 min.	20 min.	120 min.	60 min.
Repeat baseline measurements					
Percent of total number of independent baselines:	5%	5%	5%	10%	10%
Satellite constellation mask					
Minimum mask angle, degrees above local horizon:	15	15	10	18	15
Minimum number of satellites observed during 75% of occupation:	5	5	4	5	5
Maximum PDOP during 75% of occupation:	5	5	5	4	5
Antenna setup					
Max. centering error (measured and phase center):	3 mm	5 mm	7 mm	5 mm	5 mm
Independent plumb point check required:	Y	Y	N	Y	N
Max. height error (measured and phase center):	5 mm	5 mm	5 mm	3 mm	5 mm
Number of independent antenna height measurements per occupation:	2	2	2	2	2
Photograph (close up) and/or pencil rubbing required for each mark occupation:	Y	Y	N	Y	N

Table 3 **Planning and Data Collection**

The use of a validation network has been proposed by others (Cramer, Wells, Vanicek and Devlin, 1990) as a method of qualifying equipment and procedures for a given accuracy classification. The merits of this proposal should be obvious. independent justification must be provided for the observation and processing procedures used on each high-production GPS geodetic control survey. The validation must be accomplished under controlled, but similar conditions to the subject work, and must utilize the same equipment and processing software. This can be accomplished by including three-dimensional control stations (e.g. HPGN or HPGN-D) at a separation typical for the project and sufficient in number to provide a reasonable sample. The existing stations must be known to one Accuracy Band better than that intended for the project. Other possibilities include separate verification on a known three-dimensional test network, or direct comparison with high-precision static observations. A validation network is the preferable method of justification for the GPS error modeling. Any augmentation to the GPS variance/co-variance statistics must be the same in the validation network as used in the minimally-constrained and over-constrained adjustments, or an explanation provided as to why different error models were justified.

DATA PROCESSING

Within the scope of these specifications, GPS data processing includes the review and cataloging of collected data files, processing phase measurements to determine baseline vectors and/or unknown positions, and performing adjustments and transformations to the processed vectors and positions. Each step requires quality control analysis, using statistical measures and professional judgment, to achieve the desired level of confidence. Each of these steps is also very dependent upon the measurement technique, the GPS receiver and antenna types, the observables recorded, and the processing software. The following "rules of thumb" have been extracted from the experiences of this committee, the recommendations of manufactures and colleagues, and review of reports and professional journals:

1. Initial Position Accuracy

The point position (absolute) coordinates of the initial station held fixed in each baseline solution must be referenced the datum for the satellite orbits (While technically WGS84 is the datum of the GPS, NAD83 is accepted as identical for these purposes) and must be known, horizontally and vertically, to the accuracy listed in Table 2. With selective availability in effect, this most likely requires that at least one three-dimensional coordinate be known for each project, even if only a local accuracy classification is being considered. As a rough estimate, a 10-meter error in the initial station coordinates will produce a 1 ppm error in the computed baseline.

2. Orbit Ephemeris

Broadcast orbits may be used for all baseline processing with lengths under 50 km. A precise ephemeris is required for high-precision differential positioning and is recommended for relative positioning baselines longer than 25 km intended to meet accuracy bands better than 0.050 meter. A combined orbit error of 20 meters has been shown to produce approximately 1 ppm error in the computed baseline. The broadcast ephemeris error is estimated to be approximately 20 to 200 meters. Since selective availability also contains GPS clock dithering which further impacts the baseline solution, arbitrary acceptance of the broadcast ephemeris is ill-advised.

3. Atmosphere Error Reduction

A standard model for ionospheric group delay and tropospheric zenith delay, using broadcast coefficients may be used for all baseline processing. Ionospheric modeling for L_1 carrier phase measurements has shown reduction in the group delay of 50 to 60 percent. Remaining unmodeled error do to group delay is expected to be 1 to 2 ppm. Ionospheric-free processing using a linear combination of L_1 and L_2 carrier should be considered for baselines over 25 km. A highly active ionosphere can have more severe implications for GPS observations. During magnetic storms, the highly-charged ionosphere can cause signal distortions making high-precision surveying difficult if not impossible due to the excessive noise and loss of signal lock. The prudent GPS surveyor would be wise to have knowledge of the current solar cycle and predicted magnetic disturbances. Tropospheric modeling using estimated coefficients for zenith delay, or possibly even watervapor measurements, may be necessary for the highest possible vertical accuracies. Repeat station observations, combined with loop closure test and least squares adjustments should identify the need for any additional tropospheric analysis. An additional satellite frequency to be used for atmosphere error reduction has been proposed for future GPS satellites. Until such time as this or other measures are implemented, caution must be exercised to avoid adverse periods of atmospheric disturbance.

4. Baseline Processing

For all Accuracy Bands better than 0.050 meters, precise relative positioning from double difference carrier phase data requires successful resolution of the integer ambiguities. Verification of the integer bias terms must be secured for all double difference fixed solutions. Given adequate satellite geometry, real number estimates of the integer biases will usually approximate integer values. Integer search techniques provide the most probable values from a selection of candidates. Relative strength of the integer selections can be analyzed by evaluation of the a-posteriori normalized sum of the square phase residuals (in cycles) from the first choice candidate versus the second choice candidate (Langley, 1984). The RMS (in cycles) for the phase residuals and the variance of unit weight should be examined to add to the level of confidence in the integer ambiguity resolution. Phase residuals on individual satellites and over specific time periods can often help to isolate inferior data. Simultaneous phase observations rejected for a solution should be less than 10 percent of the total observations. The processing software must be capable of producing from the raw data the relative position coordinates and the corresponding variance/co-variance statistics for input to the three-dimensional adjustment program.

5. Least Squares Processing

Least squares adjustments must be performed for the final data analysis and coordinate determination. The software used must be capable of computing formal a-priori standard errors from the baseline variance/co-variance statistics, and must use models which account for the reference ellipsoid for the network control, orientation and scale differences between the GPS and network control datum. The software must also be capable of including the network control constraints as weighted observations. GPS error modeling must accommodate that portion of the error estimates generated by the baseline processing software, realistic estimates for centering error, antenna height and phase center stability, and the differences between different types of baseline solutions. Since error modeling will be different for different types of observations, variance groups of different observations (e.g. rapid static, kinematic, terrestrial, L_1, ion-free) should be assembled and their weights established and analyzed independently. When a scaler is used to augment the matrix elements of any variance group, the same values must be used for all similar baseline solutions, and the same values must be used in the over-constrained adjustment as were used in the minimally-constrained adjustment and the validation network adjustment. Any such scaler or modification of the variance/co-variance matrix must be clearly documented in the project report.

Orthometric height determination for projects exceeding 10 km^2 in area or 10 km in any dimension must include modeled geoid heights in a two-height or collocation adjustment mode. Single-height and two-height mode adjustments for geoid modeling should not be used to compute horizontal coordiantes for the project. Seperate horizontal and vertical adjustments, with a minimum of constraints in the opposing deminsion, should be used to avoid inappropriate application of rotation and scale parameters.

6. Kinematic Initialization

Since intermittent kinematic processing requires integer bias terms throughout each observation period, provisions must be made to generate integers at the beginning and verify them at the end of each kinematic session. This is most effectively accomplished by antenna swaps, static observations, fast ambiguity resolution and/or on-the-fly ambiguity resolution at the beginning and end of each kinematic session.

7. Least Squares Adjustment Analysis

Careful analysis of the minimally-constrained and over-constrained least squares adjustments is the key to completing a project which meets its intended objectives. The following statistics must be evaluated for each adjustment performed on a given project:

- The network variance of unit weight (variance factor) and degrees of freedom, for all variance groups, must be evaluated. A variance factor of less than 1.5 and approaching 1.0 is considered a conservative statistic for geodetic control surveying.

- The RMS, minimum and maximum values, and the standard deviation of the absolute observation residuals should be equal to or better than the desired Accuracy Band. The sign of residuals at a particular point or network loop may indicate an undetected systematic error or blunder in the observations which is being smeared across that portion of the network. Any common trends in observation residuals is cause for further examination.

- The standardized residuals, computed from the absolute residuals divided by their respective propagated standard error, must be compared against the Chi-square test and Tau Criterion. Standardized residuals exceeding the Tau value should be further investigated as possible outliers. Failure to pass the Chi-square test is an indication that some or all of the a-priori errors have been improperly modeled. These tests are also highly influenced by external network constraints. It is not uncommon for good-quality networks to fail Chi-square and isolated residuals exceeding the Tau Criterion are not necessarily cause to reject the adjustment. These are strict statistical tests which should be used as tools to arrive at a final network adjustment meeting the objectives of the project.

- Any significant changes between the statistics from the minimally-constrained adjustment and the over-constrained adjustment should be investigated.

- Posteriori errors must be computed at the 95 percent (2 sigma) confidence level for the adjusted station coordinates and for the relative positions for all adjacent station pairs. Error ellipse provide a useful evaluation of the station confidence. Nearly circular and uniformly small error ellipse are an indication of a well-conditioned network. Irregularly shaped or unusually large error ellipse indicate problems with the satellite constellation used, gross errors or a weakness in the network design.

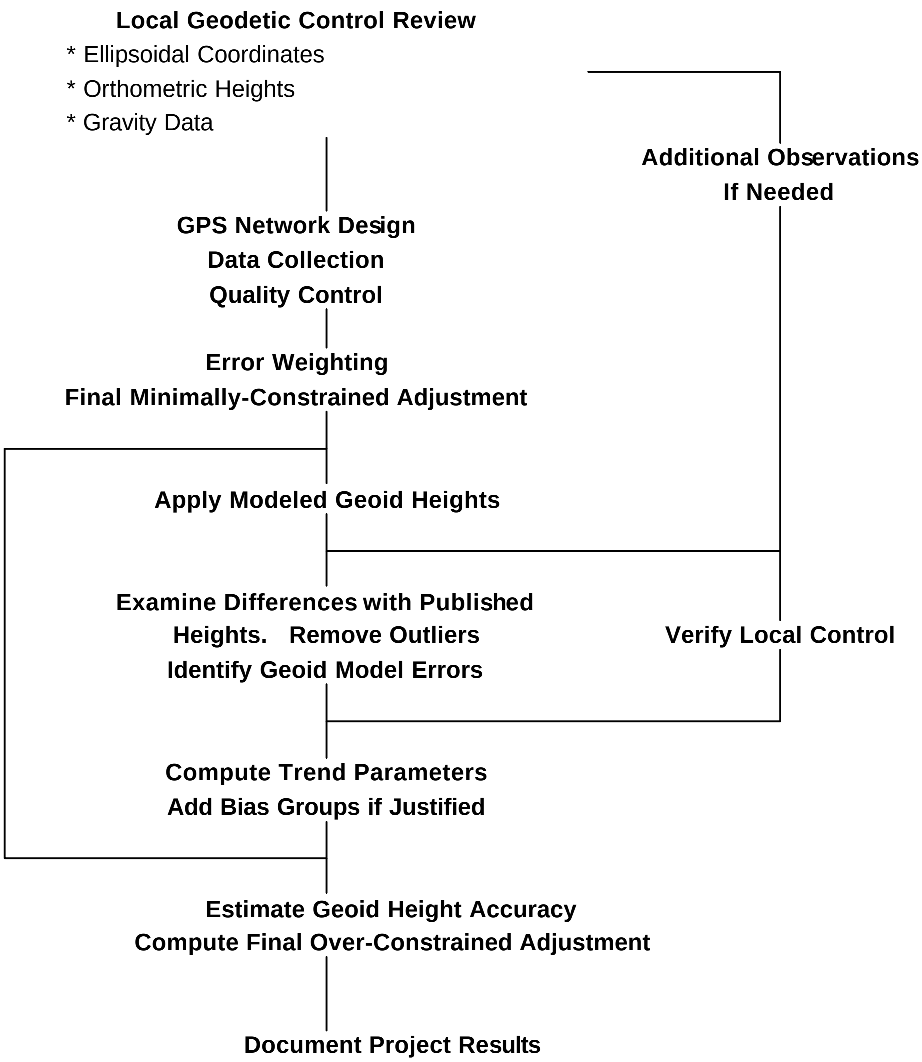
- As described in the Accuracy Standards section herein, apparent position shifts must be computed between those obtained from the minimally-constrained adjustment and known values at the external network control. This is used as a check for systematic errors prior to computing any appropriate orientation parameters for rotation and scale and assigning provisional accuracy classification(s) for the project.

- Error ellipse computed from random error propagation from a least squares adjustment, where the survey measurements have the correct weights, and where the horizontal and vertical datum values are weighted using one-sigma network accuracies of the existing control, are used to evaluate the accuracy of a project. Relative error ellipse values are used to compute error circle radii for Local Accuracy classification. The evaluation must be made from each point to all adjacent points regardless of the direct connections in the network. Point error ellipse are used to compute error circle radii for Network Accuracy classification.

GEOID MODELING

Within the context of these specifications, geoid modeling involves preanalysis to evaluate control requirements, observation planning to mitigate model deficiencies, and least squares adjustments to compute network orientation parameters and final orthometric heights. The following chart illustrates the process for orthometric height determination from GPS geodetic control networks.

Figure 1 **Orthometric Height Determination Process**



Three strategies for geoid modeling are considered in these specifications. Each employs least squares adjustment methodologies for orthometric height determination using a previously quality control checked GPS network. The network must be free of blunders and systematic errors leaving only analysis of the local vertical control and geoid undulations to be resolved. No changes to the GPS error weighting is permitted within the geoid modeling process. The following three least squares adjustment methodologies are described:

1. Single-Height Mode

    A single-height mode adjustment is accomplished by constraining the GPS network to three or more known orthometric heights and computing rotations and scale to accommodate the geoid separation and geoid slope. Single-height mode, termed the "Scale and Rotation Method", is fully described in (Vincente, 1987). In this scenario geoid slope is absorbed by rotations about the X and Y axis of a horizon-based coordinate reference and the geoid height is absorbed in a scale factor. The single-height mode relies upon the existence of a uniform geoid slope throughout the project area. For this reason the network must not extend over any known or suspected inflection points in the geoid surface. This procedure is limited to networks of limited extent - less than 10 km^2 and less than 10 km in any dimension.

    While three known elevation constraints can define a plane representing the geoid slope, a minimum of four constraints are required to provide one degree of freedom for the rotation and scale parameters.

2. Two-Height Mode

    A two-height mode adjustment is performed where the adjustment software is capable of adjusting the GPS vectors on the reference ellipsoid while importing geoid heights from a grided geoid model to maintain the relationship between the GPS network adjustment and the known elevation constraints. The imported geoid heights are held as fixed "place keepers" between the two reference surfaces. In theory elevations can be obtained for all unknown points in a network based upon a single known height. These computed elevations however, would be limited by the uncertainties in the geoid model and the error present in the single constraint. Redundant network design tells us that three to four known heights are the minimum required elevation constraints.

    The two-height mode allows network size to be increased to much greater areas since undulations are accounted for by the gridded geoid model. Orthometric height constraints must be strategically placed to identify any rotations needed to correct the modeled geoid surface. Individual adjustments for bias groups may be needed to apply different rotations to portions of the network lying in different tilt planes.

3. Least Squares Collocation

    Least squares collocation, or integrated geodesy, is an approach where dissimilar observation types (e.g. GPS vectors, leveling observations, modeled geoid heights) are combined as weighted observations in a network adjustment. In this process, interpolated geoid heights from a gridded model are allowed to adjust along with the GPS vectors and any terrestrial observations. The adjustment software must be capable of developing sophisticated models which account for the correlations between individual observations, such as how much an adjustment in one geoid height should affect the adjacent geoid heights, and between different observation types. One difficulty lies in establishing the relative weights for the different observations.

Bias groups can be accommodated by corrector surfaces included in the geoid height model eliminating the need for separate adjustments for each bias group. Unless the corrector surface has been precisely determined previously, redundant othometric height constraints are needed to compute rotations and verify the local control.

Each of the above described strategies requires a fully verified three deminsional network meeting all the specifications for the desired level of ellipsoid height accuracy. This includes examination of repeat baseline measurements, loop closures, absolute and standardized vector residuals, and differences with published coordinate values. The network design process and preplanning is critical to avoiding the pitfalls in geoid modeling. As illustrated in the process chart, inconsistancies in the local vertical control network due to subsidence, disturbed monuments, or disimilar control sources must not be allowed to contaminate the computed trend parameters. Failure to exercise extreme caution in this step can introduce significant errors into the computed heights. For instance, a local area of subsidence, if not detected, could be entirely absorbed in within the rotation parameters for the bias group. Errors in the geoid model or GPS ellipsoid heights could be similarly masked. Herein lies the greatest difficulty in GPS derived orthometric heights. The analysts must identify the magnitude of vertical discrepencies and apply corrections to the geoid model and vertical constraints which are appropriate to its source. This can only be accomplished with abundant levels of redundancy and careful analysis. The following table provides additional requirements specific to the geoid modeling process.

Table 4 **Geoid Modeling Requirements**

Spatial Accuracy Classification	Orthometric Height		
	0.005-0.020 meters	0.020-0.050 meters	0.050-0.500 meters
Bias Zones			
Maximum size in area (km sq) and any dimension for a bias zone used to determine orientation parameters for the geoid model			
Using single-height mode:	-----	10km/10km	10km/10km
Using two-height mode:	10km/10km	100km/10km	200km/20km
Using integrated geodesy:	10km/10km	100km/20km	1000km/100km
Datum Constraints			
Minimum number of known and verified elvations on the same datum within each bias zone.			
Using single-height mode:	-----	5	4
Using two-height mode:	5	4	4
Using integrated geodesy:	5	4	4
Seperate adjustments required for horizontal and vertical networks.			
Using single-height mode:	-----	Y	Y
Using two-height mode:	Y	Y	N
Using integrated geodesy:	N	N	N
Geoid Height Values			
Additional datum constraints required in all 1 min. cells of gravity voids of ___ or greater.	1	1	3
Additional datum constraints required at all inflection points in geoidal undulations.	Y	N	N

DOCUMENTATION

Geodetic control surveying in California is considered the practice of Professional Land Surveying (B&P, Chpt. 15, Art. 3, Sec. 8726). As such, all geodetic surveying projects in the State, except as specifically exempted by the law, must be performed under the responsible charge of a Professional Land Surveyor or Professional Engineer licensed to practice land surveying in California.

1. A written project report must be prepared, signed, and sealed by the licensed professional in responsible charge of the geodetic control surveying. The final report must be submitted as documentation of the successful completion at the conclusion of the geodetic control project. Included as a minimum should be the following:

    a. A narrative description of the project which summarizes the project conditions, objectives, methodologies, and conclusions.

    b. Discussion of the observation plan, equipment used, satellite constellation status, and observables recorded.

c. Description of the data processing performed. Note the software used, the version number, and the techniques employed including integer bias resolution, if applicable, and error modeling.

d. Provide a summary and detailed analysis of the minimally-constrained and over-constrained least squares adjustments performed. List the observations and parameters included in the adjustment. List the absolute and standardized residuals, the variance of unit weight, and the relative confidence for the coordinates and coordinate differences at the 95% (2 sigma) confidence level.

e. Identify any data or solutions excluded from the network with an explanation as to why it was rejected.

f. Include a diagram for the project stations and control at an appropriate scale. Descriptions for each for the monuments should be included in hardcopy or digital form.

2. Data files, including observations, computed baselines, adjustments, and coordinates, if not submitted with the project report, should be archived for inspection and future analysis.

3. Documentation for the procedures and processing validation network must be included in the project report. The same documentation may be used on more than one project with similar conditions.

SUMMARY

Two issues stand out as particularly significant within these specifications. First, least squares analysis is the primary process by which the stated project conclusions are justified. However, this process is only valid when sufficient redundancy is provided and correct assumptions have been made regarding the probability of errors. Second, the processing of raw GPS observables has been the subject of much innovation and experimentation. This is a trend which is certain to continue and whose course is difficult to predict. It is the responsibility of the professional in charge to employ those techniques which are appropriate for the subject project and to provide verification that the stated conclusions are valid. These specifications have been written with a deliberate bias toward conservatism. This bias should be endorsed by the GPS practitioner's own work, since to do otherwise is to jeopardize quality and feign professionalism.

This committee is sincere in its interest to promulgate realistic guidelines and specifications to assist high-quality geodetic control surveying. Revisions and modifications to these specifications are expected. The concerns and experiences from interested colleagues will aid in this process.

REFERENCES

Bock, Yehuda, June 1991. "Continuous Monitoring of Crustal Deformation", *GPS World*, Pg .40-47

Braum, John, Chris Rocken, James Johnson, July 1994. "Consistency of High Precision GPS Antenna", UNAVCO/UCAR Technical Report

California Business and Professions Code, Professional Land Surveyors Act, Chapter 15, Article 3, Section 8726

Craymer, M.R., D.E. Wells, P. Vanicek, R.L. Devlin, December 1990. "Specifications for Urban GPS Surveys", ACSM *Surveying and Land Information Systems*, Vol. 50, No. 4

Federal Geodetic Control Committee, September 1984. *Standards and Specifications forGeodetic Control Networks*, U.S. Department of Commerce

Federal Geodetic Control Committee, August 1, 1989. *Geometric Geodetic Accuracy Standards and Specifications for Using GPS Relative Positioning Techniques*, Version 5.0, U.S. Department of Commerse, May 11, 1988, Reprinted with corrections: August 1, 1989

Federal Geodetic Control Subcommittee, October 4, 1994. *Standards for Geodetic Control Networks*, Draft version, U.S. Department of Commerse

Frei, Erwin, G. Beutler, 1990. Rapid Static Positioning Based on the Fast Ambiguity Resolution Approach: The Alternative to Kinematic Positioning"

Klobuchar, John A., April 1991. "Ionospheric Effects on GPS", *GPS World*

Langley, R.B., G. Beutler, D. Delikaraoglou, B. Nickerson, R. Santerre, P. Vanicek, D. Wells, September 1984. "Studies in the Application of GPS to Differential Positioning", University of New Brunswick, Technical Report 108

Leick, Alfred, June 1993. "Accuracy Standards for Modern Tree-Dimensional Geodeitc Networks", ACSM *Surveying and Land Information Systems*, Vol. 53, No. 2, Pg. 111-116

NAVSTAR-GPS Joint Program Office, February 1993. *NAVSTAR GPS User Equipment Introduction*, Public release version, Pg. 12-27 through 12-32

Remondi, Benjamin W., 1986. "Performing Centimeter-Level Surveys in Seconds with GPS Carrier Phase: Initial Results", Proceeding of the Fourth International Geodetic Symposium on Satellite Positioning, Austin Texas, Pg. 1229-1249

Snay, Richard A., July 26, 1993."HTDP User's Guide", Software version 1.5

Vincente, T., 1987. "On the Use of GPS Vectors in Densification Adjustments", *Surveying and Mapping*, Vol. 47, No. 2, Pg. 103-108

Zilkoski, David B., December 1993. "NGS/CALTRANS San Diego GPS-Derived Orthometric Height Cooperative Project", Draft Report