

# GNSS Surveying Standards and Specifications, 1.1

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Joint Publication of:

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# GNSS Surveying Standards and Specifications

Joint Task Force of:  
California Land Surveyors Association  
California Spatial Reference Center



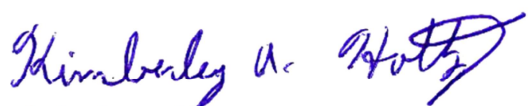
December 10, 2014

Under the direction of the Board of Directors of the California Land Surveyors Association, and the Executive Committee of the California Spatial Reference Center, the undersigned performed research, queried subject matter experts and the professional community, and over the period of February through November of 2014, prepared the following GNSS Surveying Standards and Specification, and Best Practices. It is our professional opinion that the requirements and recommendations therein describe appropriate definitions and practices for geodetic control surveying in compliance with the California Business and Professions Code § Chapter 15, Professional Land Surveyors Act, Section 8726(f), and the California Public Resources Code § Division 8, Chapters 1, 4 and 5, California State Plane Coordinates, California Geodetic Coordinates, California Orthometric Heights, Sections 8801-8819, 8870-8880, and 8890-8902.

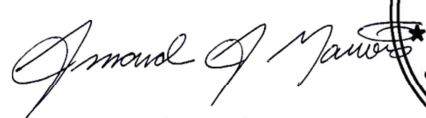
  
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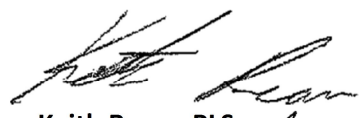
  
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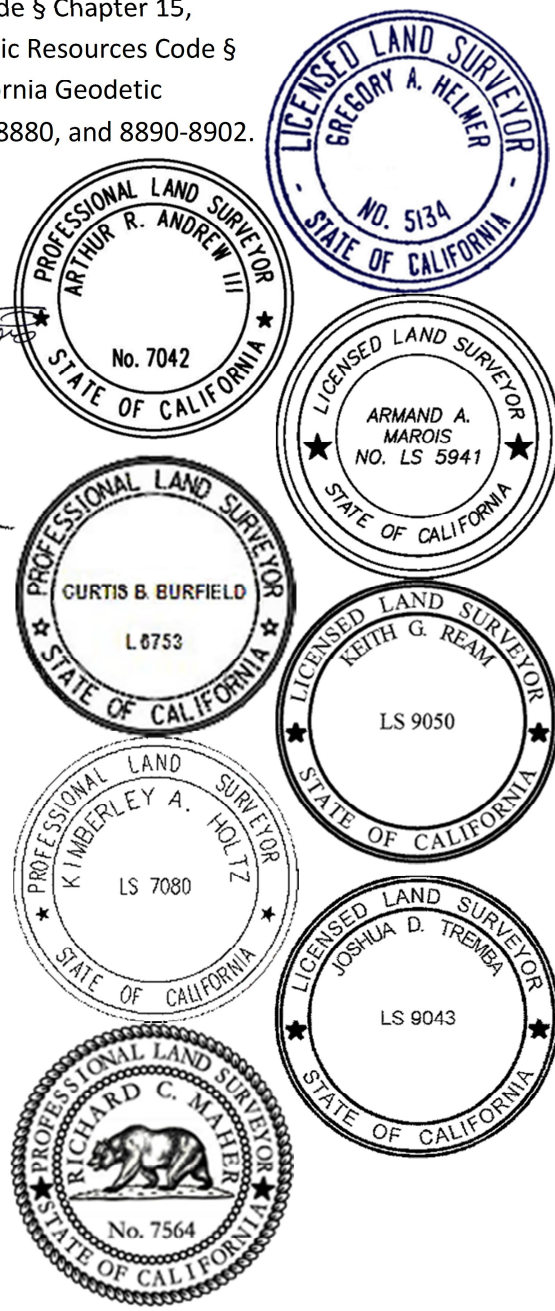
  
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## INTRODUCTION

In the three decades that geodetic surveyors have been employing Global Navigation Satellite Systems (GNSS) for precise positioning work, the technology has profoundly influenced the practice of land surveying. Beyond that, GNSS has entered the common lexicon and become pervasive with sophisticated positioning capabilities in our cars and cell phones, and GNSS enabled mapping sensors piloting our streets and navigating our skies. The U.S. Department of Labor <sup>(2005 i)</sup> identified geospatial technology as one of the top three emerging technologies for the 21st century workforce. *“Because the uses for geospatial technology are so widespread and diverse, the market is growing at an annual rate of almost 35 percent, with the commercial subsection of the market expanding at the rate of 100 percent each year.”* The motivations to become spatially enabled will continue to drive innovation and application into the foreseeable future. Surveying and mapping will continue to be one of its beneficiaries, and the profession stands in a unique position to promote high-quality practices, particularly in the establishment of geodetic control.

Interestingly, GNSS innovation for surveying and mapping has been less about improved accuracy or precision and more about faster and more easily validated positioning, and to some extent, decreasing costs. It seems that accuracy needs within the surveying and mapping community for the most part have been met, while more and faster data collection is yet an unfulfilled demand.

This publication was born out of a similar recognition in 1995 when the California surveying community found a need to promulgate specifications for geodetic control surveying using the emerging capability of positioning with GPS in a kinematic or moving mode. The resulting document, *“Specifications for Geodetic Control Networks Using High-Production GPS Surveying Techniques”*, Version 2.0, July 1996 <sup>(1996 i)</sup>, found its way into technical presentations and professional services contracts, and is still without fatal flaw, though seriously dated in the context of the above noted innovations. The California Land Surveyors Association and the California Spatial Reference Center authorized a joint task force to update, and rewrite as necessary, the document. Hence portions of the content herein are borrowed completely from the 1996 document, while others are addressing newer concepts and understanding, and the publication is arranged in a strategically different presentation.

Standards are presented first as the definition of specific accuracy classifications, and the testing methods, proofs and documentation necessary to achieve acceptable geodetic control work. This section is modeled from Federal Geographic Data Committee publications, and is similarly an outcome-based standard, open to current or future technology or methods able to withstand the required level of proof. Best Practices are presented after Standards as an expression of current geodetic control surveying capabilities, and the procedures widely recognized as capable of achieving stated levels of accuracy as defined. Best Practices provide the professional with guidelines known to produce high-quality work, but are not a mandate for methods and processes.

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 and testing to clearly demonstrate the stated accuracy.
- Adequate data processing and analysis.
- Sufficient documentation to allow verification of the results.

The standards and specifications enumerated herein are directed at these concerns. These standards and specifications are intended to be used as quality control for GNSS geodetic control surveys in Accuracy Classifications 0.5 cm to 10 cm. While higher accuracy has been demonstrated using similar techniques, 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:

- Achievement of accuracy standards by the survey's results, including sufficient independent testing and proof.
- Adherence to the Best Practices where applicable, or validation of deviations or alternate methods.
- Preparation and archiving of documentation showing compliance with these standards, specifications and best practices.

Whereas the documentation requirements are the same for any survey, certain aspects of the standards and 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 standards and specifications will contribute to this process.

## STANDARDS

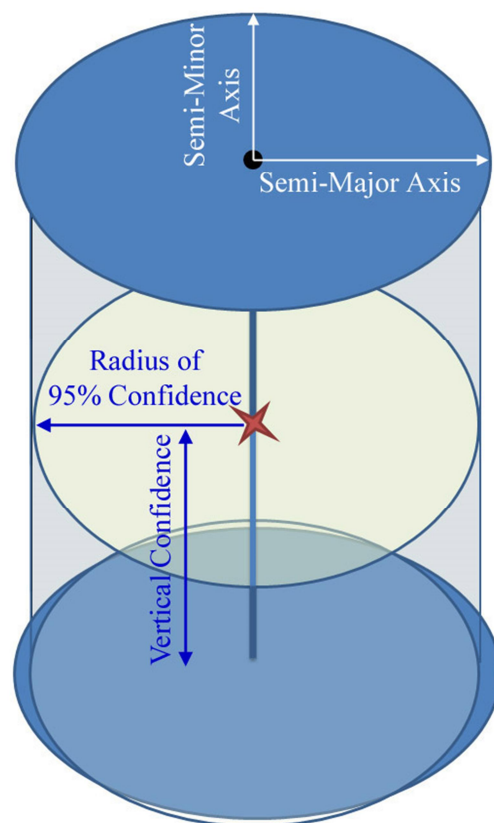
### Horizontal and Vertical Accuracy Standards

As previously stated, these standards and specifications accept the spatial accuracy standards published by the Federal Geographic Data Committee (FGDC), specifically, *Part 2: Standards for Geodetic Networks*, FGDC-STD-007.2-1998<sup>(1998 ii)</sup>. Horizontal and vertical positioning accuracies are to be qualified to positions evaluated at the 95% confidence level ( $-2\sigma < \mu < +2\sigma$ ). Horizontal accuracies are reported as the length of the radius for a circle where the estimated location of the point would have a 95% probability of being within. Similarly, the vertical accuracy is to be reported as the distance +/- from the estimated location of the point that would have a 95% probability of being within. Accuracies must be reported differently for horizontal and vertical dimensions, but may be reported for each network point or generalized over a project. For most practical purposes, the semi major axis of the 2-sigma error ellipse can be adopted as the radius length. Precise computation of the radius of the 95% confidence error circle is a double integral function which may be approximated by the product of the semi major axis and a third-degree polynomial of the ratio of the semi major to semi minor axis. This approximation including the polynomial coefficients, which typically results in a slightly smaller value than the semi major axis, is published in the Institute Of Navigation journal<sup>(1985 i)</sup>.

The volume space for positional accuracy is a simplified model representing horizontal and vertical confidence. It is centered on the adjusted point, scaled to 95% confidence, and bounded by upper and lower planes of vertical confidence of a given height component and the radius of the horizontal confidence.

For most practical purposes, the semi-major axis of the error ellipse can be adopted as the horizontal radius of confidence.

Since horizontal and vertical are correlated dimensions derived from the GNSS Cartesian coordinate frame, the cylindrical volume space is statistically conservative.



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These standards and specifications address procedures for achieving classifications 0.5 cm to 10 cm, the upper and lower margins of which reach the practical limitations for which these procedures were designed. Classifications less than or equal to 2 mm or greater than 100 mm, while perfectly valid for classification of horizontal and vertical positions, are considered to be outside of the scope of these standards and specifications and therefore not considered herein. The following lists the accuracy classifications for geodetic control. Horizontal position, ellipsoid height, and elevation (i.e. geopotential height or orthometric height) classifications retain the same nomenclature although each is classified independently.

Classification	95% Confidence Region		Notes
	Meters	Feet	
1-Millimeter	≤ 0.001		Outside the scope of these specifications.
2-Millimeter	≤ 0.002		
<b>0.5 cm</b>	<b>≤ 0.005</b>	<b>≤ 0.016</b>	<b>Horizontal and vertical accuracy classifications included in these specifications.</b>
<b>1 cm</b>	<b>≤ 0.01</b>	<b>≤ 0.033</b>	
<b>2 cm</b>	<b>≤ 0.02</b>	<b>≤ 0.066</b>	
<b>5 cm</b>	<b>≤ 0.05</b>	<b>≤ 0.164</b>	
<b>10 cm</b>	<b>≤ 0.1</b>	<b>≤ 0.328</b>	
2-Decimeter	≤ 0.2		Outside the scope of these specifications.
5-Decimeter	≤ 0.5		
1-Meter	≤ 1		
2-Meter	≤ 2		
5-Meter	≤ 5		

Five steps are necessary to classify a geodetic control network:

1. The survey measurements must be documented and made available for validation. This includes data files, notes, and other field evidence.
2. A minimally-constrained least squares adjustment is required. Blunders and systematic errors must be eliminated and an observation weighting strategy applied and validated.
3. Error propagation in a constrained least squares adjustment is used to compute provisional network accuracy values for individual points. Network accuracies (one-standard-deviation uncertainties) of the known control must be included in the error propagation computations.
4. Comparison between minimally-constrained values and known control must be made, and if not within the desired accuracy classification, additional scrutiny is necessary.
5. 95% confidence error ellipse values, and vertical confidence intervals augmented as necessary to account for geoid height uncertainty, are computed for all network points from the final constrained least squares network adjustment, and the accuracy classification established from the maximum values within the network or sub-set (bias group) thereof.

Comparison of the absolute residual values to the desired and claimed accuracy is recommended as an additional important quality assessment. While accuracy classifications are used to specify accuracy for a network or sub-network, reporting on the actual 95% confidence regions for each individual station is valuable metadata that should be reported.

### *Network versus Local Accuracy Standards*

Two types of accuracy classification are in the FGDC publications <sup>(1998 i)</sup> <sup>(1998 ii)</sup> and are described herein. Network accuracy is intended to measure the relationship between the control point in question, and the realization of the datum. Local accuracy measures the positional accuracy relative to other points within the same network. Both accuracy standards are computed by random error propagation from a least squares adjustment, where the survey measurements have the correct weights, and where constrained datum values are weighted using one-standard-deviation network accuracies of the existing network control. The concept of network and local accuracy is fairly intuitive, but the implementation is not quite as clear.

Random error propagation from a least squares adjustment where the constraining values are weighted according to the network accuracy published for the control, is used to compute 95% confidence regions (i.e. ellipse and height bars) for the network points, together with relative confidence regions between immediately adjacent points. Unless error ellipses are elongated, in which case observation data should be suspect regardless, the semi-major axis may be adopted as the horizontal network accuracy for a given point. The largest relative confidence region between adjacent points may be adopted as the local accuracy for a given point. Significant variation in the relative confidence among all adjacent points should be cause for further investigation.

Local accuracy is intended to quantify the repeatability that a surveyor should expect when measuring between two adjacent points. In practice, the assessment typically results in small difference from the network accuracy value. For this reason, network accuracy is adopted herein as the most intuitive and useful metric for classification of geodetic control accuracy. Reporting on local accuracy is useful information that should be considered for thorough documentation beyond the project's accuracy classification.

Propagated error ellipse may be used directly from the constrained least squares adjustment for horizontal accuracy assessment. Similarly propagated height bars may be used directly for accuracy assessment of ellipsoid heights, and for geopotential heights as proposed by the NGS Ten-Year Strategic Plan <sup>(2013 i)</sup> where the elevation definition is strictly from mathematical model. Orthometric heights, such as NAVD88, or derivation of tidal datum, require the height bar as propagated by least squares, to be augmented with the estimated uncertainty of the geoid model, as discussed further in Height Determination herein.



## Testing Procedures and Methods

Accuracy assessment for geodetic control surveys is performed as described above using random error propagation from a least squares adjustment, analyzed at the 95% confidence level. This assessment is only valid if four vital conditions have been met and validated:

1. All observation errors must have been reduced to only random and normal uncertainties. Any systematic errors, blunders, or miss-closures outside of a normal distribution, invalidate the statistical premise of least squares, and therefore must be eliminated.
2. The observations must have sufficient redundancy to isolate outliers and distribute random errors. Care must be taken to ensure that observations are truly independent in their redundancy, not merely in quantity.
3. A valid weighting strategy, including processing statistics and appropriate augmentations as discussed in Adjustments, must be developed and applied to the observation data.
4. Testing of the final results must be addressed by auditing an independently-determined sample, comparison to existing data, or internal analysis performed independently (e.g. different software and/or observables). Where not practical, testing should be addressed in explanation for why it was not performed, and perhaps project equipment and procedures tested and compared to a known validation network.

Independent testing from a source of higher accuracy is the preferred test for all forms of geospatial data <sup>(1998 iii)</sup>. Geodetic control surveys are typically performed to some of the highest levels of accuracy, so ANSI and FGDC recognize that internal evidence in the form of repeated measurements and redundancy are often the most practical method of proof. This does not mean however, that independent testing cannot be employed as part of the testing and validation for the project. Some of the possible testing procedures that should be considered include:

- Including existing control published to a higher or equivalent accuracy level. Verifying closures on published values in a minimally-constrained adjustment is a great quality control testing process, and particularly critical in height determination from GNSS data.
- Independent point determinations from data collected for significantly longer observation periods. This could be a replicated sub-network within a project, or independent solutions from independent processors such as OPUS or one of the Precise Point Positioning (PPP) services. Care must be exercised to apply appropriate datum and velocity transformations so that an equivalent comparison can be made. Using different software for the testing also adds an additional level of redundancy.
- Integrating specific quality control points such as a known distance calibration line or a series of test points set on an optically-straight line.
- Sufficient redundant measurements analyzed independently for closure and repeatability.

## **Documentation**

Geodetic control surveying is protected as the practice of Professional Land Surveying in California (Cal. BPC § Chapter 15, Article 3, 8726(f)). 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.

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 narrative description of the project which summarizes the project conditions, objectives, methodologies, and conclusions.
- Discussion of the observation plan, equipment used, satellite constellations and status, and observables recorded.
- 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.
- 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% confidence level.
- Describe the methodology and results of the independent testing procedures.
- Identify any data or solutions excluded from the network with an explanation as to why it was rejected.
- Include a diagram for the project stations and control at an appropriate scale. Descriptions for each of the monuments should be included in hardcopy or digital form.
- Listing of the final coordinate values, regardless of the format (e.g. printout, ASCII file, CADD, GIS database) must always include the necessary metadata, specifically: horizontal and vertical datum, epoch and datum tag, coordinate projection name and/or parameters, units, and accuracy classification claimed. A simple listing of North,East,Elevation,Feature should never be transmitted as the results from a geodetic control survey.

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

## **BEST PRACTICES**


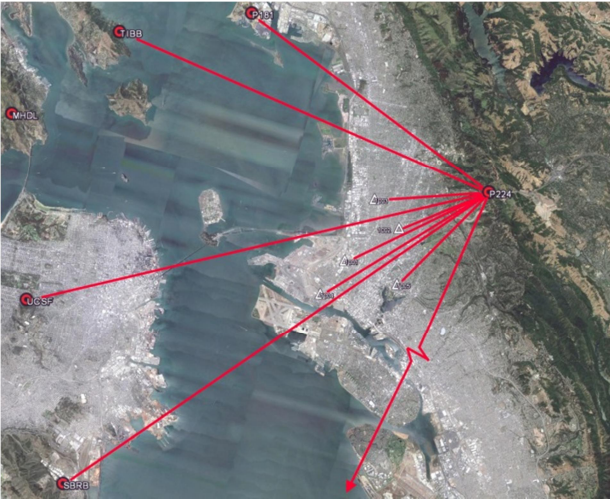
### **Network Design**

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, greatly influencing the success of any network design, will be discussed under Data Collection.

Space-based measurement systems such as GNSS, are not significantly affected by such factors as network shape, as viewed planimetrically, or the intervisibility of stations on the ground. This provides the opportunity to focus upon the availability of the project control for constraints, the intended outcome of the survey, and geometrically sensitive issues such as geoid modeling and control station velocities. Rather than being limited to the classical surveying control notions such as strength of figure, more applicable to the “direct” measurement of angles and distances, efficiencies and an increase of resulting accuracies can be maximized by the selected measurement techniques and network design. Network design certainly does have relevance for the elimination or reduction of potential error sources, for providing adequate ties to the existing geodetic reference system, and establishing the resulting accuracies. These goals should be the primary focus of the design over less influential ideals of a visually recognizable conventional survey network.

There is currently not a single or proven “best” method of network design and as of the writing of these specifications, experts in the scientific and academic arenas, as well as recognized experts in the application of GNSS technologies in geodetic control, are re-evaluating past taught practices in light of concepts such as network versus local accuracy. It is the intent of these guidelines to allow for this continued evolution in our understanding of the effect of network design, bring it to the reader’s attention, and not restrict or limit with rigid specifications. With the goals of achieving the standards listed herein Horizontal and Vertical Accuracy Standards, addressing the requirements for the identification and reduction or elimination of error, providing for the redundancy required for statistical analysis of a stochastic measurement system, heretofore considered unconventional network designs may quite simply become the “best” methods.

Geophysical issues (e.g. geoid modeling, station velocities) that are particularly sensitive to the distribution of observations and network constraints together with spatial issues such as the availability of reliable control, proposed and existing station locations, network size, and the intended accuracy standard, contribute in such various ways to make a single proposed network arrangement or measurement methodology insignificant. The ideas that follow provide considerations for the application of GNSS technology to control network design. The intent is to encourage the practitioner to determine applicability, and especially to note and discuss the decisions made along with the results within the project reporting. The following illustrates two broad types of network design with advantages and disadvantages noted.

Loop Network Design	Hub Network Design
	
<p><u>Advantages:</u></p> <ul style="list-style-type: none"> <li>▪ Loop closures provide meaningful sub-network analysis tool.</li> <li>▪ Network diagrams appear more rigorous.</li> <li>▪ Can be useful to distribute distortions in the local geodetic control system.</li> </ul> <p><u>Disadvantages:</u></p> <ul style="list-style-type: none"> <li>▪ Imposes artificial correlations between observations.</li> <li>▪ Sacrifices precision to gain assessment of adjacent stations.</li> </ul>	<p><u>Advantages:</u></p> <ul style="list-style-type: none"> <li>▪ Maintains precision of GNSS observations.</li> <li>▪ Processing software may be able to leverage short and long lines to improve atmospheric modeling.</li> </ul> <p><u>Disadvantages:</u></p> <ul style="list-style-type: none"> <li>▪ Distortions in the local geodetic control system must be rigorously modeled to avoid depositing in vectors adjusted back to the hub(s).</li> <li>▪ Network constraints demand more rigorous analysis.</li> <li>▪ Edge matching to adjoining networks is more challenging.</li> </ul>

### Monumentation Standards

When performing geodetic control surveys, the type of marker and/or monument selected for each survey should be tailored to meet the minimum accuracy requirements for the individual project. See U.S. Army Corps of Engineers Manual *EM 1110-1-1002, Survey Markers and Monumentation* <sup>(2012 i)</sup>, Fig 3-1 and 3-2 for guidelines comparing the types of monuments with the positional accuracy of the survey.

### Monuments

The following monuments types are recommended:

1. Stable three-dimensional (3D) monuments approved by NGS <sup>(2011 i)</sup>, or any existing NGS or USGS approved 3D monuments. Generalized as:  
 NGS 3-D Deep Driven Rod Marks (AKA Class A)  
 Concrete Monument With Disk

Survey Disk in Structure or Rock mass

2. Monuments Types “A” through “C” and “G” as described in US Army Corps of Engineers Manual EM 1110-1-1002 <sup>(2012 i)</sup> Generalized as:
  - Type A – Deep Rod with 3-foot finned section
  - Type B – Stainless steel Deep Rod with sleeve
  - Type C – Disk in rock or precast concrete structure
  - Type G – Disk in cast-in-place concrete monument base
  
3. Monuments “A”, “B”, and “D”, or equivalents, as shown in Caltrans Standard Plans 2010, Drawing A74 <sup>(2011 ii)</sup>. Generalized as:
  - Type A – Disk in cast-in-place concrete monument base
  - Type B – Concrete monument and disk in monument well
  - Type D – Concrete monument and disk in monument well
  
4. Manufactured control monuments, at least 30” long, such as:
  - Bernsten© Top Security Rod Monuments
  - Surv-Kap© Aluminum Rod Monuments
  - FENO© Survey Monuments
  
5. Galvanized Iron Pipes, minimum size of 2” X 24”, or 1” X 30”, with brass or aluminum disk cemented or epoxied to the top of the pipe (minimum disk diameter equal to or greater than the outside dimension of the pipe). The bottom of the pipes must be set at a minimum of 30 inches below grade.

### *Site Selection*

Select a secure location that will provide natural protection, such as one well away from traffic, and out of the path of future development, when possible. Provide for monument stability by selecting a location that reduces the influence of soil conditions. Avoid setting monuments in low, potentially wet areas, on slopes, or on any fill material. Crests of hills are generally good locations as they reduce the effects of frost heave, and the consistency of the soil tends to be more firm. The best sites are those with public access, such as within the rights of way of public streets and highways.

Sites should be free of vertical obstructions blocking the horizon such as buildings, overhangs, terrain, trees, fences, utility poles, overhead lines, or any other visible obstructions. A non-obstructed sky 15 degrees above the horizon is best. Monuments should be in safe relatively-traffic-free locations, and away from trees, signs, buildings, chain link fences, or any other objects that may block the satellite signals or create the conditions for multi-path.

## Data Collection

A thorough operations plan should be the primary concern when beginning the data collection portion of a high-accuracy geodetic control network. The most effective methodology must be planned and executed in order to minimize expenses, and insure that the project objectives are achieved. Data collection is often the most costly project element in terms of labor, as well as health and safety risk to operators and the exposed public. Given that GNSS is capable of very high accuracies, it is sometimes enticing to create a network that far exceeds the accuracy requirements for a given project. Therefore, when planning and executing the data collection portion of a high-accuracy control network it is best to find a balance between the desired end results and the minimum amount of redundancy needed to achieve the intended results. That being said, achieving this perfect balance between efficiency and accuracy can only be accomplished through experience and sound professional judgment due to the extreme variations in job conditions.

The purpose of including sufficient redundancy in a network is to remove all blunders and systematic errors, and to provide proof of the precision to which a measurement is made thereby ensuring that random errors are below appropriate threshold for the project.

- **Blunders** are defined as unintentional human error; examples include: setting up on the wrong point, incorrectly measuring the antenna height, or applying the wrong geodetic datum or epoch date.
- **Systematic Errors** are cumulative under the same conditions; examples include: improper antenna phase center, inaccurate rod level adjustment, or commencing vector processing from an inaccurate position. To a degree, multipath, clock and orbit biases, ionospheric and tropospheric conditions can be considered systematic and subject to similar minimization techniques.
- **Random Errors** are unpredictable in sign or magnitude from the observation data. Random error may be within the tolerances of the project and therefore appropriate for network adjustment, or may exceed the tolerances for the project and demand elimination similar to the treatment of systematic errors.

Observing GNSS at different sidereal times, with different satellite configurations, and different atmospheric conditions, is the strongest defense against systematic errors and excessive random errors. Two independent observations in reasonable agreement have a high likelihood of minimal systematic and random error, with increased ability to isolate blunders. This technique will be referred to as repeat station observations for the purposes of this document. The minimum sidereal time displacement between repeat station observations are listed in the Data Collection table. If there is a large time difference (weeks or months) between repeat station observations, the 4 minute negative time offset for sidereal time should be accounted for with subsequent observation times. For example, the satellite geometry for a session recorded at 11:00AM will be nearly identical to the satellite geometry at 9:00AM 30 days later. Therefore, 9:00AM must be used when calculating the sidereal time offset for the second station observation.

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It is also prudent to have different surveyors conduct the repeat station observations, especially if they are using different equipment. This further increases redundancy and also allows the processor to more easily find evidence of blunders. This is because of the possibility that a surveyor may set up on the wrong mark, consistently measure the wrong antenna height, or use equipment that is out of adjustment. If this same surveyor makes the initial station observation and then all subsequent repeat station observations on a given point, then there is less ability for an independent check on the set-up. In this scenario it is likely that the erroneous elevation or horizontal position (or both) will make its way to the final coordinate report.

Using sound survey techniques is imperative to achieving the desired accuracies in a network. Therefore, each piece of equipment to be used must be checked and adjusted before and after the survey. The maximum allowable centering error must be well below the intended final accuracies, as described in the Data Collection table. Random observation errors must be estimated and properly weighted when performing the least squares adjustment. It is strongly recommended that fixed height tripods be used for all precise vertical control applications, thereby eliminating one of the most common human error sources in antenna height measurement. If a slip-leg tripod must be used, the antenna height must be measured before and after each observation session in both feet and meters. When using fixed height tripods with rotating center poles, the bubble should be rotated and checked over a 180 degree arc at each setup. It is also wise to keep the level bubble out of direct sunlight for a period of time before checking it, no matter what setup is being used. The minimum test schedules are:

1. For Accuracy Classifications of 2 cm or less, each centering device should be tested prior to a survey's commencement and as soon as possible after a survey's completion. For long-term projects, testing must occur at regular 30-day intervals
2. For Accuracy Classifications greater than 2 cm, each centering device should be tested within 6 months prior to a survey's commencement and within 6 months after a survey's completion.

Consistent and accurate checking and subsequent documentation of antenna heights to the ARP (antenna reference point) greatly reduces the chance of introducing systematic error into a network. NGS has released absolute antenna phase-center models for nearly all commercially available antennas. By properly using these phase-center models when processing, phase center drift can be nearly eliminated. The models are referenced to the ARP, thus emphasizing the importance of proper field procedures.

A mission plan including a schematic and schedule for each day of work will help survey crews perform in the most efficient manner possible. Travel time, set up time, and time for unforeseen events must be considered when creating a mission plan. Various online utilities are available which predict satellite visibility and DOP (dilution of precision) at a given location and time series. These utilities, combined with the use of NOAA space weather prediction services can help greatly in the mission planning process, and further increase efficiency. Predicted DOP and satellite visibilities should be considered when planning session occupation time.

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For each observation session, accurate documentation is required. This documentation must include at the minimum:

1. The beginning and ending times for occupation
2. The name of the surveyor performing the occupation
3. The receiver and antenna identifiers (make, model, serial number)
4. The centering device 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. Digital photograph of monument if required
9. Any problems encountered during the logging session

The duration of any GNSS 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. Although not offered as specific criteria, the GNSS Positioning and Processing Methods Definitions in the Appendix herein, provides typical observation times for various methodologies. 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, Data Collection table, lists additional recommendations for planning and data collection.



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<b>Data Collection</b>	<b>Horizontal</b>	<b>Horizontal</b>	<b>Horizontal</b>	<b>Vertical</b>	<b>Vertical</b>	<b>Vertical</b>
<b>Spatial Accuracy Classification</b>	<b>.5 cm-2 cm</b>	<b>2 cm-5 cm</b>	<b>5 cm-10 cm</b>	<b>.5 cm-2 cm</b>	<b>2 cm-5 cm</b>	<b>5 cm-10 cm</b>
<b>Repeat Station Observations percent of number of stations</b>						
Two times:	100%	100%	80%	100%	100%	80%
Three or more times:	10%	10%	0%	50%	25%	0%
Sidereal time displacement between occupations (start time to next start):	60 min.	45 min.	30 min.	120 min.	60 min.	45 min.
<b>Satellite Observations</b>						
Minimum number of satellites observed during 75% of occupation:	7	6	5	8	7	5
Maximum PDOP during 75% of occupation:	3	4	5	3	4	5
<b>Antenna Setup</b>						
Maximum centering error (measured and phase center):	3 mm (0.016')	5 mm (0.016')	7 mm (0.023')	5 mm (0.016')	5 mm (0.016')	7 mm (0.023')
Independent plumb point check (rotating plummet, 2 <sup>nd</sup> level bubble, etc.):	Y	Y	N	Y	N	N
Maximum height error (measured and phase center):	5 mm (0.016')	5mm (0.016')	5 mm (0.016')	3 mm (0.010')	5 mm (0.016')	5 mm (0.016')
Number of independent antenna height measurements per occupation:	2	2	2	2	2	2
Digital Photograph (location and close up) for each mark occupation:	Y	Y	Y	Y	Y	Y
Fixed Height Tripod Recommended:	N/A	N/A	N/A	Y	Y	Y

**Data Processing**

Within the scope of these specifications, GNSS data processing includes the review and cataloging of collected data files, processing phase measurements to determine baseline vectors and/or unknown positions, performing adjustments and transformations of the processed vectors and positions. Each step requires quality control analysis, using statistical measures and professional judgment, to achieve the desired 95% confidence level. Each of these steps is also very dependent upon the measurement technique, the receiver and antenna types, the observables recorded, and the processing software. The following "rules of thumb" have been extracted from the experiences of this task force, the recommendations of manufacturers, colleagues, and a review of reports and professional journals:

### *Initial Position Accuracy*

The initial station's (absolute) coordinates held fixed in each baseline solution must be referenced to the satellite orbits datum. WGS84 as realized by IGS08 or later is the recommended datum for basing GNSS processing solutions. NAD83 may be an acceptable equivalent for these purposes, but in fact introduces a systematic error that must at least be considered. Presently, the origin of NAD83 differs by 2.2 meters (7 ft.) from WGS84. As a rough estimate, a 10-meter error in the initial station coordinates from WGS84 will produce a 1 ppm error in each computed baseline.

### *GNSS Orbits*

Extremely accurate and timely predicted and post-fit orbits and clock synchronization are readily available for the GNSS constellations, and when properly used, virtually eliminate errors associated with satellite ephemerides. Beside the standard broadcast ephemeris, there are three additional types of orbit data available that can and should be utilized to increase processing precision. Ultra-rapid is a 24-hour predicted orbit (6-hour latency) and is released four times a day. Rapid Precise has a 13-hour latency and is available the following day. Final and/or precise ephemeris is posted within 12 to 14 days. It is recommended that the Rapid or Final be used for high-precision networks and for relative positioning baselines longer than 25 km (15 miles) intended to meet Accuracy Classifications better than 5 cm. A combined orbit error of 20 meters has been shown to produce approximately 1 ppm error in the computed baseline. The broadcast ephemeris error for GPS is estimated to be less than 5 meters (16 ft.) for predicted satellite position and 5 nanoseconds in clock drift. The broadcast ephemeris may be acceptable for data processing, but its arbitrary acceptance is not recommended.

### *Atmosphere Error Reduction*

A standard model for ionospheric group delay using broadcast coefficients and established tropospheric model for zenith delay may be used for most baseline processing. Ionospheric modeling for L1 carrier phase measurements has shown reduction in the group delay of 50 to 60 percent. Remaining unmodeled error due to group delay is expected to be 1 to 2 ppm. Ionospheric-free processing using a linear combination of L1 and L2 carrier should be considered for baselines over 10 km (6 miles). A highly active ionosphere can have more severe implications for GNSS 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 GNSS surveyor would be wise to have knowledge of the current solar cycle and predicted magnetic disturbances. The Space Weather Prediction Center provides daily reports on solar and geophysical activity, including A and K indices and a subscription service for email alerts. Future additional satellite frequencies are anticipated which could further aid ionospheric error reduction. Until such time as this or other measures are implemented, caution must be exercised to avoid adverse periods of space weather disturbance.

Maintaining control-level-GNSS-derived vertical accuracies requires the application of tropospheric modeling using one of the established global models (e.g. Hopfield, Saastamoinen, etc.). Most survey-grade software packages provide for application of one or more of these tropospheric models. Near-real-time models from NOAA provide 10 km (6 miles) resolution for temperature, pressure and moisture, the contributing factors to dry and wet components of signal delay. Real-time GNSS networks have the potential to deliver localized zenith delay data, and some software is able to leverage simultaneous long and short baselines to improve tropospheric modeling. Without sophisticated localized tropospheric modeling, baselines exceeding 20 km (12 miles) should impose caution to avoid differential weather conditions. Repeat station observations, combined with rigorous height-constraint analysis in a minimally-constrained least squares adjustment should identify the need for any additional

tropospheric analysis.

### *Baseline Processing*

For Accuracy Classifications better than 10 cm, 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. 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 given 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.

### *Least Squares Processing*

Least squares adjustments must be performed for the final data analysis and coordinate determination. The software utilized must be capable of computing formal a-priori standard errors from the baseline variance/co-variance statistics, and must be able to use models which account for the reference ellipsoid for the network control, orientation and scale differences between the GNSS and network control datum. The software must also be capable of including the network control constraints as weighted observations. GNSS error modeling must accommodate that portion of the error estimates generated by the baseline processing software, plus 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 dissimilar for different types of observations, variance groups of observations (e.g. rapid static, static, terrestrial) should be assembled and their weights established and analyzed independently. When a scalar 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 constrained adjustment as were used in the minimally-constrained adjustment and any validation network adjustment used to assess network accuracy. Any such scalar or modification of the variance/co-variance matrix must be clearly documented in the project report.

### *Adjustment Analysis*

Careful analysis of the minimally-constrained and constrained least squares adjustments is the key to completing a project which meets its intended objectives. Once the minimally constrained adjustment has been performed, the surveyor should analyze the baseline residuals and statistical outputs and ascertain whether any baselines should be removed from subsequent adjustments. This process relies on the baseline network being observed with sufficient redundancy to enable erroneous baselines to be detected. 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 Classification. The sign of residuals at a particular point or network loop may indicate an undetected systematic error or blunder in an observation(s), which is being spread across that portion of the network. Any common trends in observation residuals are cause for further examination.
- The standardized residuals, computed from the absolute residuals divided by their respective propagated a-priori standard errors, should 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 constrained adjustment should be investigated.
- A-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 and a good distribution of satellites (DOP). 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 Horizontal and Vertical 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 ellipses and height bars computed from random error propagation in a constrained 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. The major axis of point error ellipse are used to compute the network accuracy classification(s).
- Relative error ellipse values between adjacent or directly-connected stations (so note method in the project report) may be used to compute local accuracy classification(s).

### Height Determination

For the purposes of these standards and specifications, geodetic heights can be divided into four geometric types each with differing considerations and methods for controlling them.

1. **Ellipsoid Heights** are the direct expression of the vertical component of geodetic positions in terms of the chosen geodetic datum. No further analysis or manipulation is needed other than a

validated constrained network adjustment, with appropriate transformations to California Geodetic Coordinates (CA Business and Professions Code).

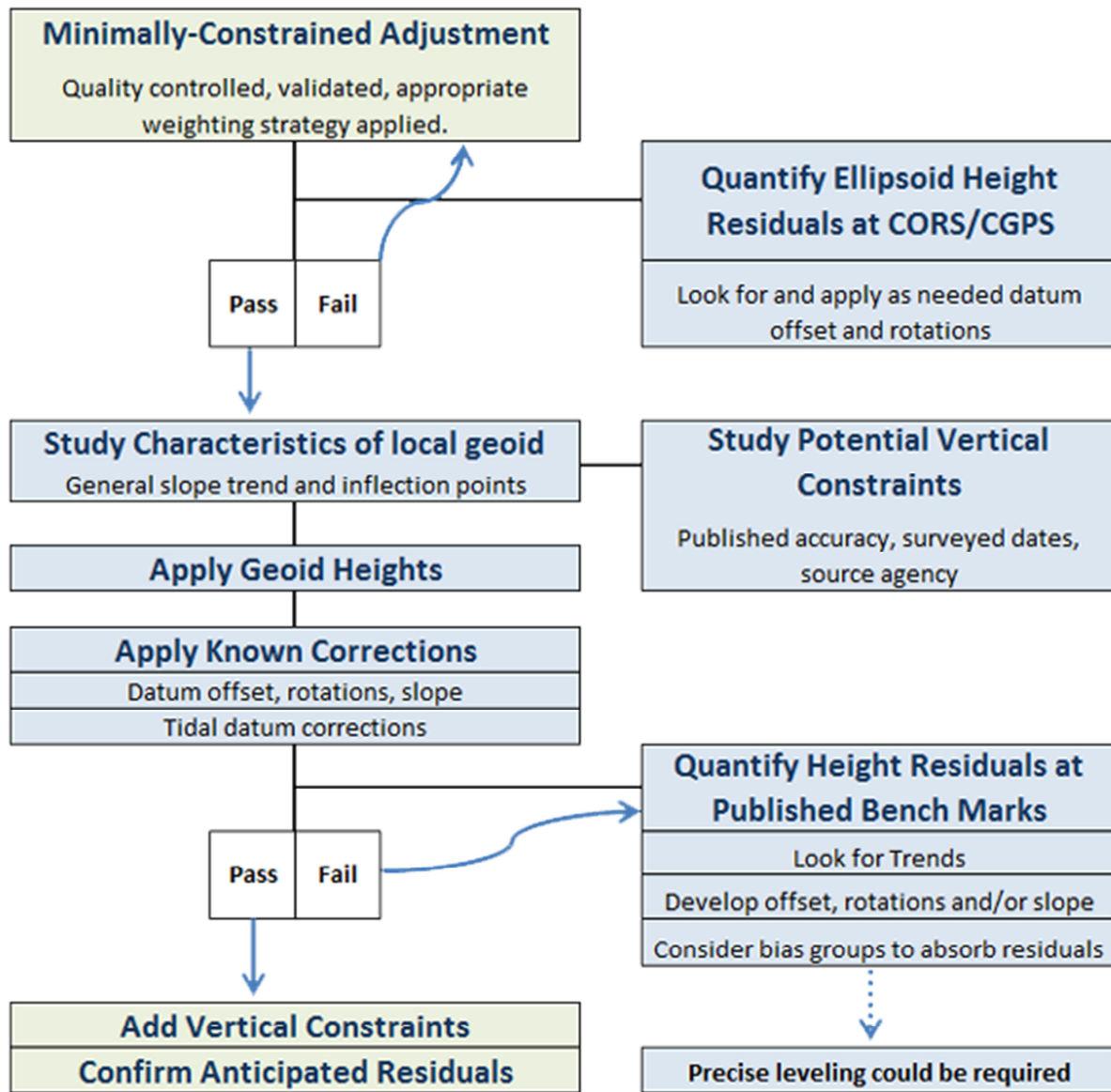
2. **NAVD88 Orthometric Heights** are in fact the realization on the ground of a national network of precise leveling and sophisticated (for the era) processing and analysis. The national network is neither geocentric nor consistent in terms of geodetic science. This presents a challenge to space-based measurement systems attempting to replicate NAVD88. Hybrid geoid models (e.g. GEOID12a) attempt to correct for inconsistencies so that reasonable approximations can be derived by simply applying the model to adjusted ellipsoid heights and verifying with sufficient observed bench marks.
3. **Geopotential Heights** at the date of this publication are yet incomplete in their definition <sup>(2013 i)</sup> but will become the official system of elevations for North America, based exclusively upon a gravimetric geoid model and adopted zero datum equipotential surface. Since geopotential heights are independent of bench marks and the realization of a national network, they require no additional analysis or manipulation. It is simply a matter of applying the appropriate geodetic models to validated ellipsoid height on the appropriate geodetic datum.
4. **Local Elevation Systems** include tidal datum (e.g. Mean Sea Level, Mean Lower Low Water, etc.), NGVD29, and some local-agency-maintained bench mark networks. Improved tidal modeling and epoch dating may eventually provide sufficiently accurate corrector surfaces for some highly-critical water bodies that would then afford direct application to geopotential heights meeting most tidal datum needs. Without a valid corrector surface, local elevation systems can only be replicated in a manner similar to NAVD88, though further complicated by the lack of a consistent geoid model to approximate elevations as a known starting point for the height determination process.

Ellipsoid heights and geopotential heights then are a straight forward process of applying the appropriate geodetic models to a validated constrained adjustment in the chosen geodetic datum. The remainder of this section will be devoted to addressing the less scientific process of derived heights in terms of NAVD88 and local elevations.

Derived heights, the terminology used herein to include NAVD88 and local elevations determined from GNSS geodetic control networks is part science and math, and part detective work. Isolating uncertainties in ellipsoid heights from uncertainties in geoid heights, from uncertainties in published bench marks rarely ends in certainty. The best advice is to keep each step in the analysis as simple as possible, working from values of highest confidence, adding components of lesser confidence one at a time running comparisons at each step to validate and edit or reject components before adding the next. The validated minimally-constrained least squares adjustment is the principal tool for analysis of derived heights. Height constraints should never be added until the geoid and any corrector surface(s), and the values of published bench marks are confirmed. By the time vertical constraints are finally imposed, the outcome will be already known and well understood. Multiple redundant bench marks, well distributed geometrically throughout the network are a necessity, with the recommended minimum being four known heights for each bias group region.

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The following process chart illustrates the recommended steps for determining derived heights from GNSS geodetic control network:



Derived heights include uncertainties in the geoid model and any correctors applied that are not accounted within the error model of the least squares adjustment, therefore additional augmentation to the propagated 95% confidence regions must be applied to properly assess the accuracy classification. NGS provides broad estimates of accuracy by region for modern hybrid geoid models. Knowledge from empirical testing on local bench marks may provide more specific assessment of the additional uncertainty contributed by the geoid model. Note that the square root of the sum of the squares of independent error sources provides the total uncertainty, i.e.  $\sqrt{\sum e_i^2}$ .

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Note: In some instances the desired vertical accuracy may only be obtained via precise differential leveling in addition to GNSS Surveying methods.

**Network Applications**

Accuracy Classification of horizontal and vertical GNSS surveying will change depending on the intended use of the GNSS survey. Also Classification would be expected to be different for the horizontal and vertical accuracies for the same GNSS survey. Following is a general table of network applications considered appropriate to the Accuracy Classifications herein.

	Accuracy Classification	
	Horizontal	Vertical
<b>Geographic Information Systems (GIS) and Asset Inventory</b>	10 cm	10 cm
<b>Planning-Level Photogrammetric Ground Control (5-foot contours)</b>	5 cm	10 cm
<b>Design-Level Photogrammetric Ground Control (1-foot contours)</b>	2 cm	5 cm
<b>Right of Way and Boundary Determination</b>	1 cm	10 cm
<b>Project Control for Design and Construction</b>	1 cm	1 cm
<b>Regional High-Precision Geodetic Horizontal Networks</b>	1 cm	2 cm
<b>Regional High-Precision Geodetic Vertical Networks</b>	1 cm	1 cm
<b>Ultra-High-Precision Networks and Deformation Monitoring</b>	0.5 cm	0.5 cm

**SUMMARY**

The foregoing Standards and Specification and Best Practices represent a consensus opinion of the authors, associated colleagues, and the two professional associations authorizing this publication. It is intended that the Accuracy Standards herein provide a common nomenclature with which agencies and institutional consumers of geodetic control can fully express by reference their intentions for control surveying projects. And collectively that this document promotes consistent high-quality and discernable GNSS geodetic control work throughout the profession. As an outcome-based standard, it is thought that these Standards will prove valid to future innovations in positioning technology. Updates and amendments, while not anticipated, should be embraced in formal republication, or in clarification, within the spirit of this original document.

## Appendix

### EXAMPLES

# Survey Report

of the  
**City & County of San Francisco**  
**2013 High Precision Network Survey**

Prepared by  
**McGee Surveying Consulting and F3 & Associates, Inc.**  
February 28, 2014

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#### SURVEYOR'S STATEMENT

This Report on the criteria, procedures and results of the City and County of San Francisco High Precision GNSS Survey was prepared by me on February 28, 2014 at the request of Bruce R. Storrs, PLS City and County Surveyor of San Francisco.

  
Michael R. McGee, PLS3945

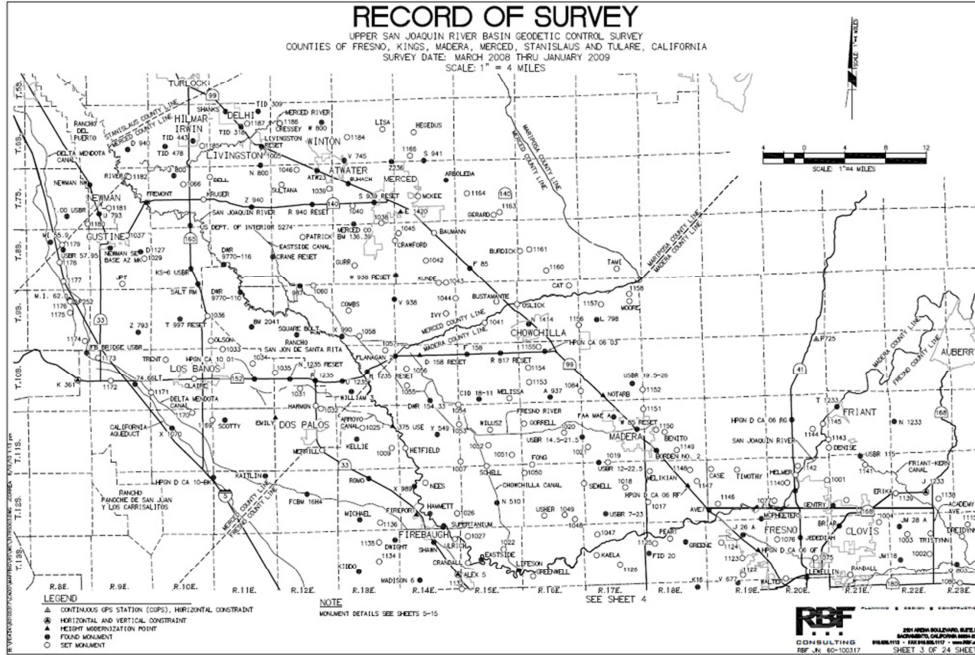


The project report should fully document the survey objectives, methods and conclusions. As a product of professional service, the report must be signed and sealed by the professional in responsible charge.



Ellipse Hgt.(ft)	Station No.	Station Name	Northing (Y) US Survey Feet	Easting (X)	Combined Factor	Elevation Feet	Station No.
1716.47	100	SET 1"IP W/METAL PLUG STAMP rbf 100' FLSH @ TO	2,119,913.17	6,341,733.11	0.9998754	1821.81	100
1343.59	101	SET 1"IP W/PP "RBF CP" FLSH NEAR BOAT RAMP	2,120,212.47	6,343,642.23	0.9998932	1448.88	101
1560.76	102	SET REBAR W/YELLOW PLASTIC CAP @ W. SIDE O	2,120,829.08	6,340,952.91	0.9998829	1666.15	102
1513.94	103	SET 1"IP W/RBF CP PLASTIC CAP @ 30' S OF DIRT F	2,118,318.58	6,341,290.92	0.9998849	1619.27	103
1549.89	104	SET REBAR W/YELLOW PLASTIC CAP RBF CP @ NI	2,118,145.03	6,339,384.35	0.9998831	1655.28	104
1353.98	105	SET CONC NAIL @ IN CONC BOAT RAMP	2,120,122.49	6,343,538.51	0.9998927	1459.27	105
1425.20	106	SET REBAR W/YELLOW PLASTIC CAP RBF CP @ O	2,121,333.61	6,338,477.51	0.9998895	1530.67	106
1241.21	107	SET REBAR W/YELLOW PLASTIC CAP RBF CP @ IN	2,124,780.78	6,339,069.56	0.9998987	1346.72	107
1363.99	108	SET REBAR W/YELLOW PLASTIC CAP RBF CP @ W	2,118,458.95	6,337,804.06	0.9998920	1469.44	108
1371.45	109	SET RBR W/YELLOW PLASTIC RBF CP @ S SIDE DI	2,125,392.03	6,348,578.70	0.9998926	1476.68	109
1525.47	110	SET RBR @ TOP OF STEEP SLOPE 500' E IF DIRT RI	2,126,575.66	6,339,387.47	0.9998854	1630.99	110
1611.73	111	SET RBR W/YELLOW PLASTIC CAP @ ON DIRT RIDG	2,127,004.33	6,341,384.74	0.9998813	1717.18	111
1442.68	112	SET RBR @ IN VALLEY 40' E OF DIRT RD	2,126,278.50	6,344,807.35	0.9998893	1548.02	112
1393.80	113	SET RBR @ ON HILLTOP ± 500' N OF VAIL LAKE	2,124,543.85	6,344,550.80	0.9998914	1499.14	113
1383.99	114	SET RBR W/YELLOW PLASTIC CAP RBF CP @ IN FL	2,124,158.34	6,346,377.48	0.9998918	1489.26	114

Coordinate lists must always include metadata for the datum, epoch date, and coordinate projection.



A control Record of Survey provides value in its public notice and its general availability to other users.

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**CONTROL SURVEY MONUMENT DATA AND RECOVERY SHEET**  
 City of Oceanside Geodetic Control Network



Number: 1008  
 Name: TREMONT ST. & GODFREY ST.



Monument Description: 2.5" Brass Disk in Curb	GPS Useable: Yes		
Stamping: LS 7854			
Center Mark: Triangle / Punch		Depth:	
Recovery By: RBF	Date: May 2013	Lid:	
Location: Top of curb in curb inlet just west of southwest curb return (+/-) 50' past the apparent intersection of Tremont Street and Godfrey Street			



California Geodetic Coordinates of 1983			CCS 83, Zone 6 Coordinates		NAD83(NSRS2007)			
Deg.	Min.	Seconds		Meters	U.S. Survey Feet	Epoch:		
Latitude:	33	10	56.07960	Northing:	613,190.352	2,011,775.35	FGDC Acc.:	1 cm
Longitude:	117	22	10.01146	Easting:	1,895,606.056	6,219,167.53	Survey by:	RBF
Ellipsoid Height (m):			-21.623	Combined Factor:	1.0000000		Adj. Date:	Oct 2013

California Orthometric Height (COH88)							
Datum	Meters	U.S. Survey Feet	Survey By	FGDC Acc.	Adj. Date	COH88	Geoid12A
NAVD88	13.096	42.97	RBF	2 cm	Oct 2013	Derived	-34.726

A database report is a good way to document all necessary station data in a single point of reference. The database is then easily integrated into an existing or proposed enterprise GIS and/or web portal.

## **GNSS POSITIONING AND PROCESSING METHODS DEFINITIONS**

Listed below are the various GNSS positioning methods that are suggested to achieve the standards that are defined within this document. Although other GNSS positioning methods could be used, these are the more common methods that are widely used with proven and achievable results utilizing the current GNSS receivers and processing software that are available today.

### **STATIC GNSS**

Static GNSS surveying is used to produce relative baseline vectors between two or more stationary GNSS units by simultaneously observing and recording data over an extended period of time (typically 30 to 120 minutes) depending on the baseline length, during which the satellite geometry changes. The observation duration should be long enough for the post processing software to resolve the integer ambiguity. This method allows various systematic errors such as delays caused by atmospheric refraction to be resolved. This method provides the highest accuracy achievable and requires the longest observation times.

### **FAST STATIC GNSS**

Fast Static GNSS surveying also known as Rapid Static, is similar to Static GNSS surveying, but with shorter observation and recording periods (typically 5 to 10 minutes). This method requires more advanced GNSS receivers (L1 and L2 frequencies) and data reduction techniques than Static GNSS. This method provides a lower accuracy than Static GNSS but requires shorter observation times.

### **REAL TIME KINEMATIC (RTK)**

RTK surveying provides the position of the rover unit computed relative to the fixed stationary base unit in real time. In this method, a communication link is maintained between the base unit and the rover unit, and the base unit streams via the communication link the pseudo-range and carrier phase measurements to the rover unit which in turn computes its position relative to the base unit. Because of the variables involved with RTK, the reliability of the positions obtained are much harder to verify than with Static or Fast Static GNSS positioning. Redundant observations are a must to produce reliable positions by averaging multiple observations obtained at different sidereal times (time staggered by 45 to 120 minutes) during which the satellite geometry changes. This method is generally limited to baseline lengths no greater than 20km due to the PPM distance related errors associated with atmospheric and satellite orbits.

### **NETWORK REAL TIME KINEMATIC (NRTK)**

NRTK surveying is the same method as RTK, but includes corrections to minimize the PPM distance related errors (atmospheric and satellite orbits) that degrade the precision of RTK, thus baseline lengths can be increased up to 50km. These corrections are computed from a network of GNSS base station units that stream their pseudo-range and carrier phase measurements to a network server

that computes a network solution. From this network solution the observation errors and their corrections are computed and broadcasted to the GNSS rover units located within the bounds on the Real Time Network (RTN). The quality of these corrections depends on the ability of the network to estimate the errors at individual base stations, as well as the capability to spatially model the errors.

### **POST PROCESS KINEMATIC (PPK)**

PPK surveys are similar to RTK procedures, but the baselines are not processed in real time. PPK involves using one or more roving units and at least one stationary reference unit over a known control point. The GNSS pseudo-range and carrier phase measurements are simultaneously collected at the reference and rover units. The data are downloaded from the receivers and the baselines processed using GNSS software. Unlike RTK, positions are computed after the field survey has been performed.

### **PRECISE POINT POSITIONING (PPP)**

Unlike relative positioning methods where the computed observation estimate is the difference in position between two or more GNSS receivers, PPP is positioning of a single receiver relative to the satellite constellation and orbital parameters. PPP relies upon precise ranging to each satellite on multiple code and carrier frequencies, including integer ambiguity resolution for each range. Sophisticated ionosphere and troposphere models are employed together with precise ephemeris data for orbits and clock drift. The multiple precise ranges are adjusted to estimate a discrete position in the satellite reference frame which then must be transformed to the target geodetic datum and epoch date.

### **OPUS and OPUS Projects**

OPUS is an online positioning users service provided by the National Geodetic Survey (NGS) that allows you to upload GNSS Static data and uses software to compute coordinates relative to the NGS Continuously Operating Reference Station (CORS) network. These resulting positions are accurate and based on the National Spatial Reference System (NSRS). OPUS requires that the antenna must remain static throughout the session, 15-minutes of data or more, antenna type, and antenna height. The final solution is delivered via email in the datum and epoch date of the CORS network. The final solution must therefore be transformed to the target geodetic datum and epoch date if different.

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