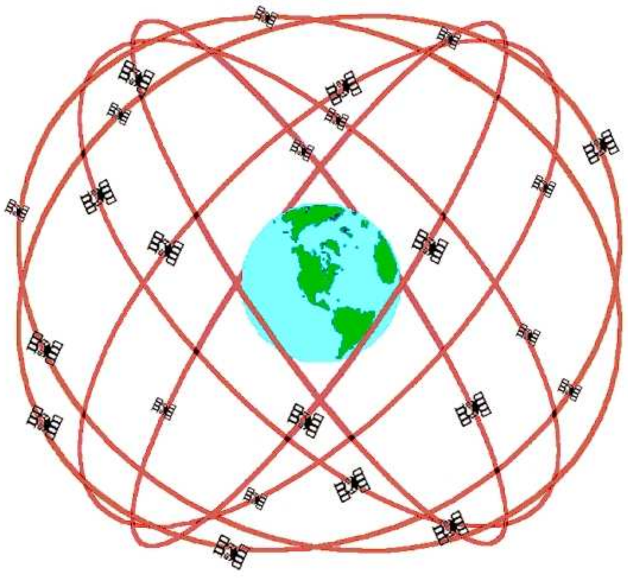
N A T U R A L   R E S O U R C E S

G P S   G U I D E B O O K

Standards and Guidelines for Land Surveying Using Global Positioning System Methods



NOVEMBER 2004

Version 1

by the Survey Advisory Board
and the Public Land Survey Office
for State of Washington Department of Natural Resources

WASHINGTON STATE DEPARTMENT OF
Natural Resources

Cover Picture “orbits” by Peter Dana 9/22/98

Table of Contents

Table of Contents
for Examples, Tables, and Figures

Examples

Tables

Figures

WASHINGTON STATE DEPARTMENT OF
Natural Resources

Doug Sutherland
COMMISSIONER OF PUBLIC LANDS

October 2004

Dear Reader:

This "Land Surveyor's GPS Guidebook" has been made possible by an incredible effort of the GPS Guidebook Committee, a diverse group of public and private land surveyors and GPS practitioners. The Committee consists of the Survey Advisory Board, representatives from the Land Surveyor's Association of Washington, the Board of Registration for Professional Engineers and Land Surveyors, as well as many other Federal, State, County, and City government agencies.

This guidebook was prepared pursuant to the authority of Chapter 58.24 RCW, which requires the Department of Natural Resources to cooperate and consult with state, county, and municipal governments and registered land surveyors for the establishment of survey standards, methods of procedure, and the monumentation of boundaries.

The GPS Guidebook Committee and DNR staff have put many hours of hard work and dedication into this project. Please give serious consideration to the information presented in this guidebook as the DNR and the surveying community work cooperatively for the benefit of the people and the industry of our state.

Sincerely,

Doug Sutherland
Commissioner of Public Lands

Foreword

The GPS Guidebook Committee is pleased to present this guidebook and present the surveying community with GPS guidelines and procedures for ***land surveying***.

We do not intend this guidebook as a complete discussion of the subject of GPS surveying or to be used as a stand-alone training tool. This introduction encourages the reader to pursue additional study. The committee members strongly believe that it is in the best interest of the surveying profession, and the public served by this profession, to incorporate this guidance into everyday practice.

The initial impetus for this guidebook came from discussions occurring during training sessions where many surveyors described how they performed GPS measurements and how they compared the ***accuracy*** of those measurements. In addition, changes in surveying equipment and procedures have shown the need for a reevaluation of current practices for performing GPS measurements. The "Surveyor's Guidebook on Relative Accuracy" was released in June 1995 and introduced the relative accuracy subject. An initial attempt was made to add a chapter to that publication, which would address the ***relative accuracy*** of GPS measurements as a practical application of the principles. That attempt was thwarted by the complexity of the subject matter. Communications with representatives from other states and from the Land Surveyor's Association of Washington emphasized the need for a cooperative learning process to make the continual transition successful.

Therefore, the committee views this guidebook as a second step in this interactive, educational effort. This guidebook has intentionally not addressed some of the newest GPS measurement technology and the committee will make future revisions as needed. We encourage your active involvement in this process. Please address any comments you may have about this guidebook or the subject of relative accuracy to staff in the DNR, Public Land Survey Office (PLSO). An address can be found at the PLSO website: http://www.dnr.wa.gov/htdocs/plso/

The guidelines[1] in this document focus on:

- Standards for Positional Accuracy
- Types of surveying that may use GPS
- Field operations and procedures
- Data processing and data analysis
- Project documentation

The use of these guidelines and the ***manufacturer's specifications*** provide a means for the surveyor to evaluate a survey and to verify that the specified accuracy standard can be achieved.

These guidelines are designed to ensure that a survey performed with GPS technology is repeatable, legally defensible and can be referenced to the ***National Spatial Reference System (NSRS)*** by addressing the following set of criteria:

1 This document will not cover geodetic control surveys and it does not cover the requirements necessary to meet the National Spatial Data Infrastructure (NSDI).

Note: Italicized words in bold font, e.g., ***independent occupation***, indicate that a definition for that word appears in Appendix A: Glossary and Definitions. (Definition words are shown **bold**, *italicized* the first instance only.)

1. Model and eliminate or reduce known and potential random components of ***systematic error*** sources.

2. Identify and minimize ***random error***s.

3. Occupational (station) and observational (baseline) redundancy to quantitatively demonstrate the required or stated accuracy.

4. Documentation of ***baseline processing***, data adjustment, and data analysis, which demonstrates the recommended procedures and required accuracy.

5. Compliance with the current Bureau of Land Management "*Manual of Instructions for the Survey of the Public Lands of the United States*."

6. Compliance with ***Revised Code of Washington (RCW)*** and ***Washington Administrative Code (WAC).***

This document is intended to provide the user with guidelines for planning, execution, and classification of surveys performed using GPS ***carrier phase*** methodology. GPS survey guidelines continually evolve with the advancements in equipment and techniques. Changes to these guidelines are expected as these advancements occur. The size, scope, and site conditions of a project may also require variations from these guidelines.

Any variations from these guidelines should be designed to meet the above criteria and to achieve the accuracy standard of the survey as suggested by this document. All variations should be documented in a project report (See Section E).

Many sources were consulted during the preparation of this document. These sources included other GPS survey standards and guidelines, technical reports and manuals (see Appendix D for References). Opinions and reviews were also sought from public and private Professional Land Surveyors who use GPS for land surveying.

All tables and references are shown in metric dimensions. Accuracy may be expressed in ***U.S. Survey feet*** units where the point coordinates or elevations are expressed in that system.

Copyright

Acknowledgements

The following people contributed to the successful completion of this Guidebook:

Representing the Survey Advisory Board for the Department of Natural Resources:
Dan Beardslee, PLS – 2004-2005 Chair – Erlandsen & Associates, Inc. – Chelan, WA
George Bradford, PLS – Cowlitz County – Kelso, WA
James Coan, PLS – 2001-2002 Chair - Renton Technical College – Renton, WA
Walter Dale, PLS – Benthin & Associates – Spokane, WA
George Ford, PLS – Longview Fibre Co. – Longview, WA
Sara Marks, PLS – 2003-2004 Chair – Spokane, WA
Jerry Olson, PLS/PE – 2002-2003 Chair - Olson Engineering, Inc. – Vancouver, WA
Jon Purnell, PLS – Peninsula College – Port Angeles, WA
Jerry Sims, PLS – Spokane County – Spokane, WA

Representing the Land Surveyor's Association of Washington:
Mike G. Mickiewicz, PLS – W & H Pacific – Bothell, WA
Larry Signani, PLS – W & H Pacific – Bothell, WA
Bruce Towle, PLS – Olson Engineering, Inc. – Vancouver, WA
James Wengler, PLS – Wengler Surveying and Mapping – Port Angeles, WA

Representing the Department of Licensing, Board of Registration for Professional Engineers and Land Surveyors:
Ron Torrence, PLS – Board of Registration – Olympia, WA

Department of Natural Resources Support Staff – Olympia, WA
David Steele, PLS – Survey Manager
Donnell Fitch, PLS – Public Land Survey Office – Manager, retired
Gwen Roy, PLS – Public Land Survey Office – Manager
Franklin Fischer, PLS – Land Survey Program – Manager

Special Review/Advisory Assistance:
Ken Bays – Bureau of Land Management – Portland, OR
Ken Chamberlain – Bureau of Land Management – Portland, OR
Jill Johnson, LSIT – Pierce County – Tacoma, WA
David George – Geoline Positioning Systems – Bellevue, WA
Susan Lundstedt – Peninsula College – Port Angeles, WA

The committee appreciates your participation.

Daniel Beardslee, PLS
Survey Advisory Board, Chair

David Steele, PLS
DNR Survey Manager

SECTION A: STANDARDS FOR POSITIONAL ACCURACY

There are a number of ***errors*** that affect the position estimates derived from GPS measurements. The errors can generally be divided into three distinct groups;

Satellite dependent errors

- ***Clock Errors***
- Satellite Orbits
- ***Selective Availability***

Propagation dependent errors

- ***Atmospheric Errors***
- Troposphere
- ***Ionosphere***

Receiver dependent errors

- ***Antenna Phase Center***
- ***Measurement Uncertainty***

The majority of the errors listed are "spatially related". In other words, two receivers at adjacent locations will experience similar errors. This characteristic is used to mathematically eliminate many of these errors by surveying with two receivers simultaneously. Because position error at both sites is assumed the same, the difference in coordinates between the two receivers should be significantly more accurate than the absolute position of each antenna phase center. This technique is termed ***differential*** positioning and is used in all surveying applications of GPS technology. Two additional sources of error that are site specific with GPS are ***cycle slips*** and ***multipath***. Cycle slips are generally caused by physical objects, which block the signal or may include seemingly insignificant things such as mesh fences or other small objects, which can momentarily block the satellite signal. Multipath is caused when the satellite signal takes two routes to the antenna phase center. One route is direct from satellite to antenna and the second generally occurs when nearby reflective surfaces deflect the signal path from the satellite to the antenna. In addition, unseen obstructions such as electromagnetic transmissions or naturally occurring atmospheric disturbances may also interfere with this signal. Ionospheric corrections should always be made on long baselines to compensate for any uncorrected Ionospheric effects on the satellite signals..

The static and ***fast static*** survey procedures in these guidelines follow long established and well-documented industry and government survey practices. Guidelines for real-time ***kinematic*** surveys are in the early stages of development. Real-time kinematic GPS is suitable for land surveying of small areas. Maintenance of ***positional confidence*** requires certain observational and occupational redundancies and checks.

Accuracy Standard

The accuracy of classical triangulation network or traverse surveys is described by a proportional standard, e.g. 1:10,000, which reflects the distance-dependent nature of ***terrestrial surveying*** error. The accuracy of GPS surveys, being less distance dependent, use different accuracy standards (linear and positional ***covariance*** standards). This use of multiple standards creates difficulty in comparing the accuracy of coordinate values obtained by different survey methods.

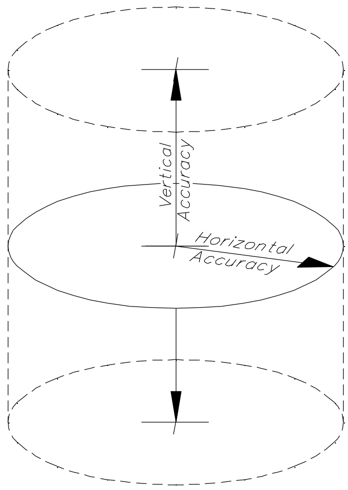
In recognition of these difficulties, the ***Federal Geographic Data Committee (FGDC)*** has changed its methodology for reporting the accuracy of horizontal and vertical coordinate values. The new reporting standard is defined by 95% confidence intervals, an ***error circle*** for horizontal uncertainty, and a linear value for vertical uncertainty.

Table 1: FGDC Accuracy Standards Horizontal, ***Ellipsoid Height***, and ***Orthometric Height***

Accuracy Classification	**95% Confidence**	**Accuracy Classification**	**95% Confidence**
1-Millimeter	≤ 0.001 meters	1-Decimeter	≤ 0.100 meters
2-Millimeter	≤ 0.002 meters	2-Decimeter	≤ 0.200 meters
5-Millimeter	≤ 0.005 meters	5-Decimeter	≤ 0.500 meters
1-Centimeter	≤ 0.010 meters	1-Meter	≤ 1.000 meters
2-Centimeter	≤ 0.020 meters	2-Meter	≤ 2.000 meters
5-Centimeter	≤ 0.050 meters	5-Meter	≤ 5.000 meters
		10-Meter	≤ 10.000 meters

All spatial data activities should use one or both of these classification schemes depending on the characteristics of the data set(s). Described below in more detail are the standards for reporting the positional accuracy for horizontal and/or vertical coordinates:

Figure 1



Horizontal Standard

The reporting standard in the horizontal component is the radius of a circle of uncertainty, such that the true location of the point falls within that error circle 95-percent of the time. See figure 1

Vertical Standard

The reporting standard in the vertical component is a linear uncertainty value, such that the true location of the point falls within +/- of that linear uncertainty value 95-percent of the time.
See figure 1

Accuracy

WAC 332-130-080 describes the use of relative accuracy in Washington State: Relative accuracy should be described in terms of ***local accuracy*** when developing a ***survey quality statement***. A quality statement appearing on a Record of Survey[2] might say, "…95% of corners shown are no greater than 0.046 m. (±0.004m) local accuracy, based upon a ***least squares adjustment*** of all traverses in the survey…". See Table 4 for expected local accuracy positional standards using various GPS methods.

Local Accuracy

Local accuracy is derived from a least squares adjustment of the survey network, which might include GPS and terrestrial data. Such a network is relative to the positions of existing ***primary control points***. These primary positions are weighted using their ***one-sigma*** network accuracies. ***Relative error ellipses*** are computed during the adjustment for all directly connected, secondary points. Each relative ***error ellipse*** is then converted to a corresponding circular measure. The average of all relative circular measures constitutes the survey's relative accuracy. Since this measurement of accuracy is based upon an average, then some measurements will be of lower quality or fall outside of the average relative error circle. Typically, most surveyors and their clients want to be assured that all their measurements meet some standard. In order to facilitate this comparison, consider the radius of a relative error circle as equal to the semi-major axis of its associated relative error ellipse. Therefore, the value of the largest relative error circle radius in the project will be the local project accuracy. See Section D: Error Analysis, for details.

Network Accuracy

The ***network accuracy*** of a control point is a value that represents the uncertainty in the coordinates of the control point with respect to the ***geodetic datum*** at the 95-percent confidence level. This is also called the ***station error ellipse***. For NSRS network accuracy classification, the datum is best expressed by the geodetic values at the ***Continuously Operating Reference Stations (CORS)*** supported by the National Geodetic Survey (NGS). By this definition, the local and network accuracy values at CORS sites are considered to be infinitesimal, i.e., to approach zero error.

Since most least squares adjustment and baseline analysis software packages express point uncertainties in the form of ***point standard deviations*** in Northing and Easting, a point's local or network accuracy can be computed by using the larger of the two uncertainties as a

---

2 That map to be filed with the County Auditor following direction found in Chapter 58.09 RCW, Surveys – Recording.

confidence circle radius. Next, these techniques can be used to compute local or network accuracies between points as desired.

Local accuracy is best utilized to check relations between nearby control points. For example, a surveyor checking closure between two NSRS points is most interested in a local accuracy measure. On the other hand, someone constructing a ***GIS*** will often need some type of ***positional tolerance*** associated with a set of coordinates. Network accuracy measures how well coordinates approach an ideal, error-free datum. For more information on local and network accuracies, consult the FGDC publication "Geospatial Positioning Accuracy Standards" (see Appendix D for references).

Network and local accuracies are fundamentally similar concepts; with local accuracy describing the positional accuracy between two points, and network accuracy describing the positional accuracy between a point and the NSRS. Used together, network and local accuracy classifications are helpful in estimating accuracies between points not directly measured, or in comparing positions determined from two separate surveys. Consider the following examples:

Example 1

Local Accuracy: If adjacent stations are positioned independently, the local accuracy between the stations can be estimated from each point's network accuracy (error circle) relative to the controlling CORS or High Accuracy Reference Network (HARN) station.
To estimate local accuracy, L_{HM} (see Figure 2):
Let N_H = the error circle at the NW corner of Section 16, and
Let N_M = the error circle at the SW corner of Section 16, then

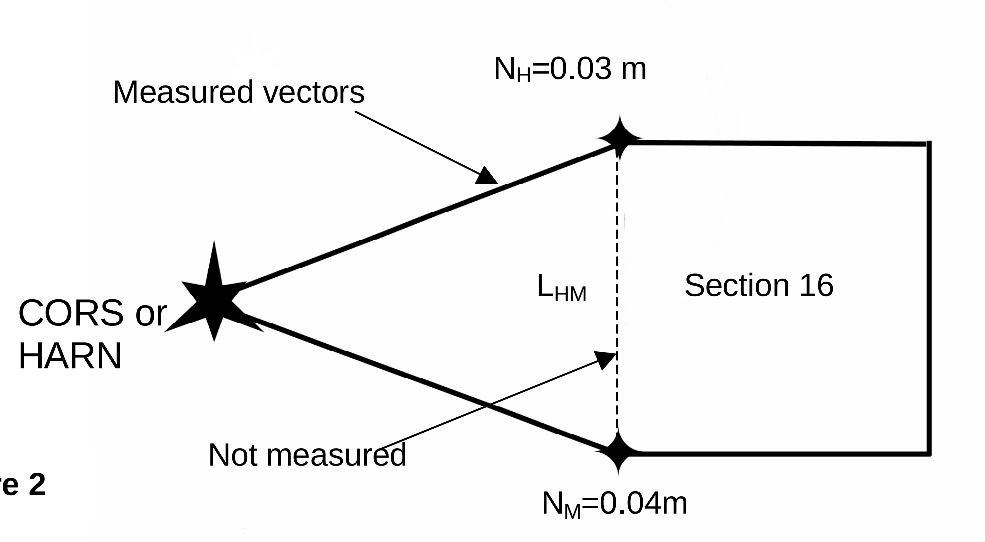


Figure 2

$$L_{HM} \approx \sqrt{(N_H)^2 + (N_M)^2} = \sqrt{(0.03m)^2 + (0.04m)^2} = 0.050m$$

Example 2

Network Accuracy: If a station has been positioned without a direct measurement from a CORS or HARN station, the network accuracy can be determined from the errors of the stations in the network.

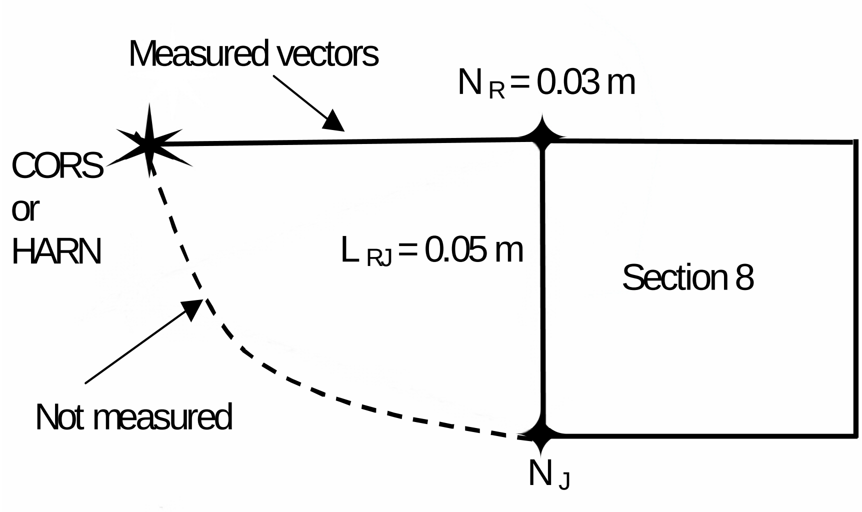
To compute network accuracy, N_J (see Figure 3)

Let N_R = the error circle at the NW corner of Section 8, and

Let L_{RJ} = the relative error between the NW and SW corners of Section 8, then

$$N_J = \sqrt{(N_R)^2 + (L_{RJ})^2} = \sqrt{(0.03m)^2 + (0.05m)^2} = 0.058m$$

Figure 3



Linear Closure Relationship

To assist in relating classical proportional standards[3] to confidence intervals, Tables 2 and 3 have been created.

In Table 2, a desired linear closure is shown across the top of the table and a distance between local survey points on the left of the table. Intersecting the column and row will show the expected local or network accuracy between the points.

Thus, two points that are 500 meters distant having a local or network accuracy of 0.05 meters has a proportional accuracy that is no better than 1 part in 10,000. Put another way, points that are to be set 500 meters apart can have local or network horizontal uncertainties of no more than +/- 0.05 meters in order to achieve a proportional accuracy of 1 part in 10,000.

3 Refer to WAC 332-130-090 Field traverse standards for land boundary surveys, which describes minimum standards for linear closures after azimuth adjustment.

TABLE 2: Linear Closure ~ Accuracy Relationship (in meters)

Distance Between Points	Linear Closure Ratio			
	1:5,000	1:7,500	**1:10,000**	1:15,000
25 m.	0.005	0.003	0.002	0.002
50 m.	0.010	0.007	0.005	0.003
75 m.	0.015	0.010	0.008	0.005
100 m.	0.020	0.013	0.010	0.007
200 m.	0.040	0.027	0.020	0.013
300 m.	0.060	0.040	0.030	0.020
400 m.	0.080	0.053	0.040	0.027
500 m.	0.100	0.067	**0.050**	0.033
750 m.	0.150	0.100	0.075	0.050
1,000 m.	0.200	0.133	0.100	0.067
1,500 m.	0.300	0.200	0.150	0.100
2,000 m.	0.400	0.267	0.200	0.133
2,500 m.	0.500	0.333	0.250	0.167

Table 3 can be used to estimate the local relative accuracy between two directly connected points. The table shows various error circles for two points, A and B. Values for point A appear in the leftmost column and values for point B appear in the topmost row. To find the relative accuracy between the points, find the error circle value for point A in the leftmost column and the error circle value for point B in the topmost row. The estimated local relative accuracy is found at the intersection of the column-row.

For example: If the expected local relative accuracy between two points is 0.050 m (3 cm & 4 cm error circles) and the desired proportional accuracy is 1: 10,000, then from Table 2, the minimum distance between the points should not be less than 500 m.

TABLE 3: Relative Accuracy between Points (in meters)

Error Circle for Point A	Error Circle for Point B									
	0.010	0.020	0.030	0.040	0.050	0.060	0.070	0.080	0.090	0.100
0.010	0.014	0.022	0.032	0.041	0.051	0.061	0.071	0.081	0.091	0.100
0.020	0.022	0.028	0.036	0.045	0.054	0.063	0.073	0.082	0.092	0.102
0.030	0.032	0.036	0.042	0.050	0.058	0.067	0.076	0.085	0.095	0.104
0.040	0.041	0.045	0.050	0.057	0.064	0.072	0.081	0.089	0.098	0.108
0.050	0.051	0.054	0.058	0.064	0.071	0.078	0.086	0.094	0.103	0.112
0.060	0.061	0.063	0.067	0.072	0.078	0.085	0.092	0.100	0.108	0.117
0.070	0.071	0.073	0.076	0.081	0.086	0.092	0.099	0.106	0.114	0.122
0.080	0.081	0.082	0.085	0.089	0.094	0.100	0.106	0.113	0.120	0.128
0.090	0.091	0.092	0.095	0.098	0.103	0.108	0.114	0.120	0.127	0.135
0.100	0.100	0.102	0.104	0.108	0.112	0.117	0.122	0.128	0.135	0.141

Example 3

Two local survey points A and B are each located by GPS with 95% certainty confidence circles. The expected relative error between the points will be as follows:

> Given confidence circles for points A and B, whose radii are 0.01 m (0.0328’). The expected random error will be:
>
> $$E = \sqrt{0.010^2 + 0.010^2} = 0.014\text{m}, \text{ or } E = \sqrt{0.0328^2 + 0.0328^2} = 0.047'$$

By substituting relative local accuracies computed with Table 3 into Table 2, the minimum distance between connected points necessary to maintain a given proportional accuracy can be determined.

Expected Local Accuracy Positional Standards for GPS Land Surveys

The following, Table 4 is provided to aid the surveyor in planning GPS surveys. It gives the smallest (minimum) and the largest (maximum) error circle expected under normal conditions. This chart can be used to pre-plan spacing on survey points when used in conjunction with Table 2: Linear Closure ~ Accuracy Relationship, in order to meet minimum standards. Care must be taken upon final adjustment of the project to assure actual results meet or exceed pre-planned expectations. Note that points measured without redundancy have "unknown" maximum error. Without redundancy, we have no assurance that points are within any specified error circle.

A least squares adjustment or other multiple baseline data analysis must be performed to produce ***weighted mean average*** point coordinates and point uncertainties in order to verify that the required level of positional accuracy has been achieved.

TABLE 4: Expected Local Accuracy Positional Standards[4]

Method	**Horizontal Max**	**Horizontal Min**	**Vertical Max**	**Vertical Min**	**Application**
Static	1.5cm	1 cm	6cm	4 cm	***Survey Project Control***, Survey Measurements
Fast Static	3cm	2 cm	9cm	6 cm	Survey Project Control, Survey Measurements
***Post Processed* Kinematic** Redundant	6cm	4 cm	9cm	6 cm	Survey Measurements
Real Time Kinematic Redundant	6cm	4 cm	9cm	6 cm	Survey Measurements
Post Processed Kinematic	Unknown	4 cm	Unknown	6 cm	Data acquisition for topographic / contour maps
Post Processed Kinematic Continuous	Unknown	4 cm	Unknown	6 cm	Data acquisition for topographic / contour maps
Real Time Kinematic	Unknown	4 cm	Unknown	6 cm	Data acquisition for topographic / contour maps
Real Time Kinematic Continuous	Unknown	4 cm	Unknown	6 cm	Data acquisition for topographic / contour maps
Differential GPS ***Resource Grade***	Unknown	See manufacturer Specifications	Unknown	See manufacturer Specifications	Resource grade mapping only

4 Surveyors are typically concerned with positional accuracy at the 95% certainty level or 2σ as shown in the table above, yet manufacturer's specifications are usually at the 68.3% certainty level or 1σ and may be obtained by dividing the minimum specification figures by 1.96.

Table 4 was created by the GPS Guidebook Committee from a variety of sources.

SECTION B: TYPES OF GPS SURVEYING

Horizontal Control

Horizontal control is a network of GPS stations tied to the NSRS or a local control scheme, which is surveyed to control all subsequent GPS or conventional survey measurements. Project control may also be used to bracket or surround a kinematic or ***RTK*** project with static points.

Using survey project control tied to the NSRS is required when a land surveyor intends to place coordinates on the final survey map following guidance in RCW 58.20.180.

The survey project control network must be established by either static or fast-static survey methods. The network measurements may be observed at the same time the survey measurements are observed and are designed to meet the following purposes:

- Provides a framework to reference the survey to a datum, a ***mapping projection***, and the NSRS.

- Supports registration of the survey measurements into the ***public record*** under RCW 58.20.180 and into a GIS.

- Serves as the basis for all subsequent, project specific GPS and conventional survey measurements.

- Ties to the NSRS will allow reporting of the network accuracy of survey measurements per "FGDC Geospatial Positioning Accuracy Standards".

- A well-designed survey project control network will offer the surveyor more flexibility when using fast static, kinematic, and RTK survey methods for the survey measurement portion of a survey. It provides an adequate amount of reference (base) station locations, ties the survey measurement points together, allows for expanding the survey area and provides accurate checks throughout the survey project.

The number of stations in the survey project control network depends upon factors such as project size, topography, positioning method used, and access. A minimum of two or more survey project control stations should be established as a reference for the survey measurements. All survey project control networks referenced to the NSRS should be tied to at least two ***High Accuracy Reference Network (HARN)*** stations (also called High ***Precision Geodetic Network (HPGN)*** stations) or two ***Continuous Operating Reference Stations (CORS)*** of the NSRS. Currently, the Washington HARN values are in ***NAD 83(1991)*** and ***NAD 83(1998)***, whereas CORS values are in ***NAD 83(CORS)*** with an epoch value like 2002.00. These adjustments should not be mixed. For further study, refer to http://www.ngs.noaa.gov/CORS/metadata1/

In the absence of HARN or CORS stations, other GPS control stations, which are referenced to the NSRS and published by or available through other federal, state, or local agencies, may be used. The use of such stations should be evaluated by the surveyor regarding the relationship to the NSRS before inclusion into the survey project control network and before any survey measurements occur. In some instances, a GPS survey may be performed without connection of the NSRS and a local or assumed datum may be utilized. Coordinate values derived from a local datum should not be published on the survey record, and the specific datum or basis of meridian should be designated. The current Washington State reference datum is the North American Datum of 1983, (see RCW 58.20.180). All control and project information should be referenced to an adjustment of NAD 83, for example, NAD 1983 (1998). Orthometric heights require reference to a datum as well. The currently preferred datum is the North American Vertical Datum of 1988 (NAVD 88) for work on land and a ***tidal datum*** with ***tide station*** reference for surveys in tidal waters. The datum tag or date of the datum represents the date of the regional least squares adjustment associated with the horizontal control point. The ***epoch*** date is used for stations in regions of episodic and/or crustal motion where the coordinates change with time. The epoch date indicates the date the published horizontal coordinates and heights are valid. All points with adjusted horizontal coordinates and/or heights that fall within a designated crustal motion region will have an epoch date based on the date of the latest survey from which the coordinates were determined.

All horizontal control networks should conform to the following:

- Be referenced to two or more NSRS or other published or local datum horizontal control stations, located in two or more quadrants, surrounding the survey project area.

- Points are connected by two or more ***independent baselines***.

- Contain valid loops with a minimum of three baselines and a maximum of 10 baselines. A loop must contain baselines from at least two different observing sessions to be valid.

- Baselines have a fixed ***integer double difference*** solution or adhere to the manufacturer's specifications for baseline lengths that exceed the ***fixed solution*** criteria.

- All stations in the survey project control network should have two or more ***independent occupations***.

- The survey project control network must be a geometrically closed figure. Therefore, single radial (spur) lines or side shots to a point are not acceptable.

Vertical Control

Based on the Federal Geographic Data Committee publication, "Geospatial Positioning Accuracy Standards" [http://www.fgdc.gov/ standards/document/standards/accuracy], guidelines were developed by the NGS for performing GPS surveys intended to achieve ellipsoid height network accuracies of 5 cm at the 95 percent confidence level, as well as ellipsoid height local accuracies of 2 cm and 5 cm, also at the 95 percent confidence level.

GPS carrier phase measurements are used to determine ***vector baselines*** in space, where the components of the baseline are expressed in terms of ***Cartesian coordinate*** differences. These vector baselines can be converted to distance, azimuth, and ellipsoidal height differences (dh) relative to a defined reference ellipsoid.

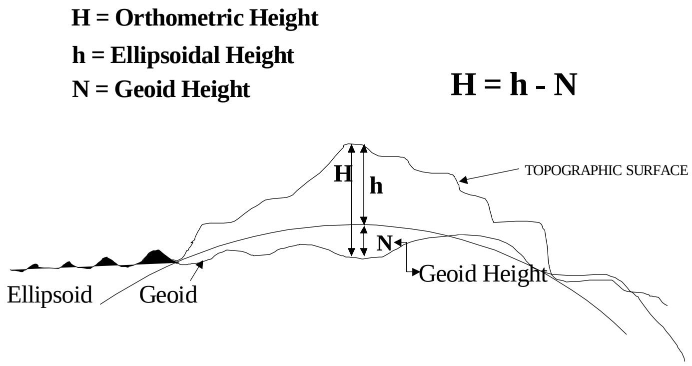


Figure 4

Orthometric heights (H) are referenced to an ***equipotential reference surface***, a.k.a. the ***geoid***. The orthometric height of a point on the Earth's surface is the distance from the geoidal reference surface to the point, measured along the plumb line normal to the geoid. These are the heights most surveyors have worked with in the past and are often called "mean sea-level" heights. Ellipsoid heights (h) are referenced to a reference ellipsoid. The ***ellipsoid height*** of a point is the distance from the reference ellipsoid to the point, measured along the line that is normal to the ellipsoid. The term ellipsoid height may be a new concept to many traditional surveyors, but has become prevalent because ellipsoid heights are derived from GPS measurements. At the same point on the surface of the Earth, the difference between an ellipsoid height and an orthometric height is defined as the ***geoid height*** (N).

Several error sources, including ***deflection of the vertical***, that affect the accuracy of orthometric, ellipsoid and geoid height values are generally common to nearby points. Because these error sources are in common, the uncertainty of height differences between nearby points is significantly smaller than the uncertainty of the absolute heights of each point. This is the key to establishing accurate orthometric heights using GPS. Orthometric height differences (dH) can then be obtained from ellipsoid height differences (dh) by subtracting the geoid height differences (dN):

dH = dh-dN

Adhering to NGS' guidelines, ellipsoid height differences (dh) over short baselines, i.e., less than 10 km, can now be determined from GPS carrier-phase measurements with **2-sigma** uncertainties that are typically better than ±2 cm. The requirement that each baseline must be repeated and agree to within 2 cm of each other, and must be repeated on two separate days during different times of the day, should provide a final GPS-derived ellipsoid height better than 2 cm at the 2-sigma level. It should also be noted that the GPS-derived ellipsoid height guidelines documented by NGS were intentionally designed to produce ellipsoid heights slightly better than 2 cm, i.e., about 1.4 cm, so they could also be used when generating 2 cm GPS-derived orthometric heights. The requirement that spacing between local network stations cannot exceed 10 km helps to keep the relative error in geoid height small, i.e., typically less than 0.5 cm. Therefore, it is possible to establish GPS-derived orthometric heights that meet accuracy standards at the 2-cm (95 percent) level routinely using GPS.

There are three basic rules, four control requirements, and five procedures that need to be adhered to for computing accurate NAVD 88 GPS-derived orthometric heights.

Basic Rules

Rule 1: Follow NGS guidelines for establishing GPS-derived ellipsoid heights when performing GPS surveys,

Rule 2: Use NGS' latest ***national geoid model***, e.g. GEOID03, when computing GPS-derived orthometric heights, and

Rule 3: Use the latest National Vertical Datum, NAVD 88, height values to control the project's adjusted heights.

Control Requirements

Requirement 1: GPS-occupy stations with valid NAVD 88 orthometric heights; stations should be distributed proportionally throughout project.

Requirement 2: For project areas less than 20 km on a side, surround project with valid NAVD 88 bench marks, i.e., minimum number of stations is four; one in each corner of project. [NOTE: The user may have to enlarge the project area to occupy enough bench marks, even if the project area extends beyond the original area of interest.]

Requirement 3: For project areas greater than 20 km on a side, keep distances between valid GPS-occupied NAVD 88 benchmarks to less than 20 km.

Requirement 4: For projects located in mountainous regions, occupy valid benchmarks at the base and summit of mountains, even if the distance is less than 20 km.

Procedures

Procedure 1: Perform a ***3-D minimum-constrained least squares adjustment*** of the GPS survey project, i.e., constrain one latitude, one longitude, and one orthometric height value.

Procedure 2: Using the results from the adjustment in procedure 1, detect and remove all data ***outliers***. [NOTE: If the user follows NGS' guidelines for establishing GPS-derived ellipsoid heights, the user will already know which vectors may need to be rejected, and following the GPS-derived ellipsoid height guidelines should have already re-observed those base lines.] The user should repeat procedures 1 and 2 until all data outliers are removed.

Procedure 3: Compute the differences between the set of GPS-derived orthometric heights from the minimum-constrained adjustment (using the current Geoid model) from procedure 2 and the corresponding published NAVD 88 benchmarks.

Procedure 4: Using the results from procedure 3, determine which bench marks have valid NAVD 88 height values. This is the most important step of the process. Determining which benchmarks have valid heights is critical to computing accurate GPS-derived orthometric heights. [NOTE: The user should include a few extra NAVD 88 bench marks in case some are inconsistent, i.e., are not valid NAVD 88 height values.]

Procedure 5: Using the results from procedure 4, perform a fully-constrained adjustment holding one latitude value, one longitude value, and all valid NAVD 88 height values fixed.

Adjustment Notes:

1. Compare repeat base lines
   The procedure is very simple: subtract one ellipsoid height from the other, i.e., the ellipsoid height from base line A to B on day 1 minus the ellipsoid height from base line A to B on day 2. If this difference is greater than 2 cm, one of the base lines must be observed again. This is a very simple procedure, but also one of the most important. Many users complain about having to repeat base lines, but requiring an extra half-hour occupation session in the field can often save many days of analysis in the office.

2. Analyze loop misclosures
   Loop misclosures can be used to detect "bad" observations. (A bad observation can include a misread antenna height, not being plumb over a point or observing the wrong point.) If two loops with a common base line have large misclosures, this may be an indication that the common base line is an outlier. Since users must repeat base lines on different days and at different times of the day, there are several different loops that can be generated from the individual base lines. If a repeat base line difference is greater than 2 cm then comparing the loop misclosures involved with the base line may help determine which base line is the outlier. According to NGS guidelines, if a repeat base line difference exceeds 2 cm then one of the base lines must be observed again, and base lines must be observed at least twice on two different days and at two different times of the day.

3. Plot ellipsoid height ***residuals*** from least squares adjustment
   Like comparing repeat base lines, analyzing ellipsoid height residuals is also important. During this procedure, the user performs a 3D minimum-constrained least squares adjustment of the GPS survey project, i.e., constrain one latitude, one longitude, and one ellipsoid height; plots the ellipsoid height residuals; and investigates all residuals greater than 2 cm. Be aware that NAD 83(1998) heights should be used since NAD 83(1991) heights are not good.

4. Select a best fit to a tilted plane
   Best fitting a tilted plane to the height differences, between GPS-derived orthometric heights and published NAVD88 heights, is a good method of detecting and removing any systematic trend between the height differences. Most GPS adjustment software available today has an option for solving for a tilted plane or rotations and ***scale parameters*** to remove the systematic trend from data if one exists. After a trend has been removed, all differences between measured and published should be less than ± 2 cm.

Survey Measurements

Survey measurements are used to define the location and extent of ownership boundaries or property lines. Survey measurements are referenced to the survey project control coordinates or by direct ties to the NSRS.

All survey measurement observations, except RTK, should conform to the following:

- Be constrained to two or more survey project or NSRS stations, which are located in two or more quadrants surrounding the survey project area.

- Points are connected by two or more independent baselines.

- Contain valid loops with a minimum of three baselines and a maximum of 10 baselines. A loop must contain baselines from at least two different observing sessions to be valid.

- Baselines have a fixed integer double difference solution or adhere to the manufacturer's specifications for baseline lengths that exceed the fixed solution criteria.

- The azimuth or bearing between any pair of stations used for reference during conventional survey measurements should be included in a network, or measured with a minimum of two independent vectors following the RTK techniques described in Section C. Baselines between station pairs must be measured for inclusion in network adjustment and analysis.

- All stations in the survey measurements shall have two or more independent occupations.

- Single radial (spur) lines or side shots to a point are not acceptable.

Data Acquisition for Topographic/Contour Maps

GPS is suited to topographic surveys for areas having large open areas for easy satellite acquisition. Some of the benefits of GPS are: 5-second occupation time per shot, the data collector is moved to each point hence it is available for recording point descriptions, and a line of sight between base and rover is not required.

Topographic surveys may be based on an existing vertical and/or horizontal datum or be totally based on an assumed datum. When based on an existing datum, ***base station*** setup procedures (see Section C; RTK System Check on page 27) must be followed for the type of data capture being used. When using an assumed horizontal and vertical datum the base point is assigned an assumed value, each shot is then based on the relative position between the base antenna and the rover antenna. For assumed datum it is recommended that at least two additional control points for the survey be set using redundant measurements to assure coordinate values are correct. Periodic checks made to these points assure that all settings and adjustments are still correct. Subsequent setups on the base location previously used as an assumed datum must be treated as an existing datum with all setup procedures followed.

When the surveyor needs assurance that objects being surveyed during a topographic survey are within a certain tolerance one should refer to Tables 4, 5, & 6 to determine if their methodology will meet their required specifications.

These surveys are usually performed using ***Post Processed Kinematic (PPK)***, Real Time Kinematic (RTK), ***Post Processed Kinematic continuous (PPK-c)***, or ***Real Time Kinematic continuous (RTK-c)*** with ***radial data capture techniques***. Vertical and horizontal project control is normally performed before topographic mapping. The setup of the base station on a known point is verified by making a measurement from the base station setup to another survey project control station or previously observed survey measurement point. If the measured x, y, z coordinate values are within the ***manufacturers' duplicate point tolerance*** of the existing coordinate values, then the setup is assumed good. The manufacturers' specifications are used to determine the expected accuracy for individual topographic points. A client may also have requirements or the surface material (hard or soft) dictates that the accuracy be greater than possible using RTK. The surveyor needs to remember that there will be a certain number of positional outliers even under ideal field conditions. Obstructions, atmospheric disturbances, solar weather, and electromagnetic forces can also cause "multipath" and "cycle slips" which will increase the number of outliers. For this reason enough positions must be acquired to negate the effects of these errors on the final map. Any feature to be mapped requiring positional certainty must be located using redundant measurement techniques.

Point positional standards are independent of, and must exceed, the ***topographic map accuracy standards*** for the map that is being created.

SECTION C: FIELD OPERATIONS AND PROCEDURES

General Requirements

The general requirements for surveys recommended by this document apply to all types of GPS surveys - static, rapid static and kinematic.

The general requirements include:

- Referring to the manufacturer's documentation for instructions as to the correct use of equipment
- All ancillary equipment such as tripods, tribrachs, optical and laser plummets, etc. should be in good condition
- Users should take extreme care when measuring the height of the antenna above the ground mark
- The point identifier should be recorded at the time of survey
- ***Satellite geometry*** as defined by the ***Geometric Dilution Of Precision (GDOP)*** should be less than 5
- All receivers must observe at least four common satellites
- The ***elevation mask*** should not be less than 15 degrees
- When establishing reference stations, marks with high quality coordinates should be adopted
- When heights are required, marks with high quality height values must be used
- Field observation sheets[5] should be used to document all static survey occupations
- It is not necessary to record ***meteorological readings***, but if desired the readings should be taken with precise instrumentation. Standard Tropospheric models should normally be used during data processing and GPS processing software will provide different model choices of which the manufacturer may provide a default model.
- Measurements for horizontal coordination purposes must form a closed figure and be connected to at least two marks with known coordinates in the desired coordinate system
- Appropriate measurement redundancy should be performed according to the type of survey

These guidelines provide a basic framework for performing surveys.

5 Documentation of the point being measured, equipment operators, equipment used to measure, time and date of measurement, antenna height measurements, and any details that may appear unusual, e.g. lightning or other severe weather.

Field Data Acquisition Methods

A variety of GPS field data acquisition methods may be used for survey measurements, survey project control, topographic measurements, and vertical control. The appropriate measurement methods are associated with these survey types in Table 5.

TABLE 5: Survey Type ~ Method Matrix

	Static	Fast static	PPK	RTK
Survey Project Control	YES	YES	NO	NO
Survey Measurements	YES	YES	YES	YES
Topographic Measurements	YES	YES	YES	YES
Vertical Control	YES	YES	NO	NO

Table 6: Summary of Carrier-Phase GPS Positioning Methods[6]

Method	Requirements	Application	Horizontal Accuracy	Comments
Static ***L1*** (Post processing)	-L1 GPS receiver -Computer for post processing -Minimum 45 minutes +	Control surveys (High-accuracy)	1 cm + 2ppm	Forgiving, tolerates cycle slips, needs single frequency geodetic receiver. Relatively long occupations; lines limited to approx 15 km, due to ionosphere
Static L1/***L2*** (Post processing)	-***Dual frequency*** GPS receiver -No ***P-code*** required -Need L1/L2 antenna	Control surveys (High-accuracy)	0.5 cm + 1ppm	+Forgiving, tolerates cycle slips, works out effects of ionosphere -Relatively long occupations
Fast-Static (Post processing)	-Dual frequency GPS receiver with either P2 or P1/P2 configuration -Need L1/L2 antenna -5 to 45 minute observation time, depending on number of ***SV***'s	Control surveys (Medium to high accuracy)	2 cm + 2ppm	+Short occupations, very efficient +No requirements for maintaining ***lock*** between points -Requires high-end receivers -More susceptible to multipath problems -Requires very careful planning or good communication in field
Kinematic Includes stop & go and continuous (Post processing)	-L1 receiver with Kinematic option -Need kinematic antenna (L1) -Data collector with survey controller software recommended 5-30 seconds average for ***stop & go kinematic*** 0.5-5 seconds average for ***continuous kinematic***	Continuous topographic surveys Feature mapping surveys	4 cm + 2ppm for stop-&-go 5 cm + 2 ppm for continuous	+Very short occupations +Most efficient data collection +One person can be the complete topographic crew -Requires ***initialization*** -Must maintain lock to 4 SV's while moving between points -Most susceptible to multipath effects -Recommended max distance between base and rover is 10 km
Kinematic Real time Kinematic (RTK) ***On-the-fly***	-L1 receiver with RTK option (includes ***RTCM*** in/out) -Need kinematic antenna (L1) -Need data links -Need data collector with survey controller software -5-30 seconds average, depending on user requirements	Hydrographic surveys (real-time, high accuracy) Location surveys	4 cm + 2ppm	+No post-processing of data required +Provides real-time coordinates +Logs vector information for network adjustment, if desired +Very efficient for both location and layout -Radio links between base and rover must be maintained (up to 5 repeaters okay) -Lines limited to 10 km
DGPS Resource Grade	L1 receiver with differential beacon, ***WAAS***, or other broadcast signal	Mapping & Location of resource features	1 m –L1 DGPS 2 m –WAAS 5 m –CA	- Not to be used for any boundary or regulatory line
Autonomous GPS	L1 receiver or ***CA code*** single frequency hand held	Orienteering Navigation	5 m-L1 30 m –CA code	

6 Surveyors are typically concerned with positional accuracy at the 95% certainty level or 2σ as shown in the table above, yet manufacturer's specifications are usually at the 66% certainty level or 1σ and may be obtained by dividing the minimum specification figures by 1.96.

Static Positioning

The static observation technique is commonly used due to its reliability and ease of data collection. Static surveys are performed by setting up equipment over ground marks at two or more stations for a predetermined period. Like all GPS surveying procedures, it is crucial that data is logged at the same time to a common set of satellites.

Static observing sessions are normally 45 minutes to 3 hours in length. Equipment manufacturers generally provide guidelines on the ***epoch interval*** and session length for various conditions. The epoch interval defines how often measurements are recorded, for example every 1, 10, or 60 seconds. A single epoch of data is sufficient to achieve centimeter-level results once the carrier ***phase ambiguities*** are resolved. The duration of static sessions must therefore be sufficient to ensure that the phase ambiguities can be resolved.

Some centimeter-level errors affecting GPS measurements tend to average out if the session is longer than 20 minutes. Where utmost accuracy is required, always collect at least 45 minutes of data.

Static positioning is primarily used for ties to the NSRS when observing survey project control. This method may also be used for the survey measurement portion of a land survey or when doing vertical control surveys.

- The minimum observation period for baselines less than ten kilometers should be 45 minutes
- The recording rate should be 5-30 seconds
- The satellite geometry should change significantly during the observation session
- Single frequency receivers may be used for short lines for non-high ***precision*** applications
- It is essential that the carrier phase ambiguities are constrained for lines less than 15km

Fast-Static Positioning

When making survey measurements using only the fast-static method, survey measurement observation schemes use either a network or multiple radial baseline approach. Use one or more reference (base) receivers, usually two, located in two or more different quadrants and one or more ***rover receiver***s with fixed height or adjustable antenna tripods/bipods. The antenna elevation on adjustable antenna tripods/bipods must be regularly checked during a survey to ensure slippage has not occurred.

Another observation scheme uses two or more receivers in a traverse or leapfrog observational scheme.

This method requires shorter occupation times (i.e. 5 to 45 minutes) than static positioning and may use a radial baseline technique, network technique, or a combination of the two. Fast static requires least squares adjustment or use of processing software capable of producing a weighted mean average of the observations. Fast-static positioning may be used for observing both the survey project control and the survey measurements of a land survey.

- baselines must be less than ten kilometers in length
- manufacturer's documentation should be consulted for determining the occupation period
- dual frequency receivers are preferred
- five or more satellites should be observed
- the recording rate may vary between five and fifteen seconds

Post-Processed Kinematic (PPK) Positioning

Satellite measurements collected at sites in the field can be stored then combined in a computer for post-processing. Results are only obtained after the fieldwork is complete. The continuous kinematic and stop-and-go ***kinematic surveying*** techniques are used to improve productivity in open areas where many points need to be located. A ***reference receiver*** is established in an open area at a fixed location. One or more rover receivers are then moved to various points of interest. With continuous ***kinematic positioning***, the relative location of the rover antenna is determined at each epoch. Whereas in the stop-and-go technique, the antenna location is only required when it is stationary over various marks.

At least 5 satellites are required to initialize and resolve the carrier phase ambiguities. At least 4 satellites and preferably 5 or more must be observed during kinematic surveys.

When making survey measurements using a PPK survey method, one should keep the project control and survey measurements separated. This method uses one or more reference (base) receivers, usually two, located in two or more different quadrants and one or more rover receivers during survey measurements.

The post-processed kinematic survey method provides the surveyor with a technique for high production survey measurements and can be used in areas with minimal ***satellite obstructions***. PPK uses significantly reduced observation times compared to static or fast-static observations. This method requires a least squares adjustment or other multiple baseline statistical analysis capable of producing a weighted mean average of the observations. PPK positioning is used for observing the survey measurements of a land survey.

- five or more satellites should be observed
- receivers should be initialized per the manufacturers recommendations
- each point should be occupied in a different session with different satellite geometry when measuring fixed points.
- the recording rate should be between one and five seconds

- single frequency receivers may be used although dual frequency receivers are preferred

Real-time Kinematic (RTK) Positioning

Real-Time ***Kinematic*** (RTK) GPS Surveys are performed with a data transfer link between a reference GPS unit (base station) and rover units. The field survey is conducted like a kinematic survey, except data from the base station is transmitted to the rover units through a data radio or through an Internet connection usually provided by a cellular link, enabling the rover unit to compute its position in real time. RTK techniques provide results with very little delay and therefore are suited for stakeout applications. Normally RTK methods are limited by the range that the rover can be from the reference. Refer to the manufacturers specifications for these ranges and corresponding precisions. RTK techniques and CORS advancements have broadened the ranges of RTK further, but these advantages are limited to regions where the technology has been embraced.

When using an RTK survey method, as the sole measurement method for survey measurement, keep the project control and survey measurements separated. RTK uses a radial style survey scheme with one or more reference (base) receivers and one or more rover receivers for survey measurements. The radial nature of RTK requires additional redundant measurements be made as part of the field survey operations and procedures, which are discussed in these guidelines.

- Five or more satellites should be observed.
- Receiver specifications should be adhered to for consistent results.
- Each point should be occupied in a different session with different satellite geometry, unless collecting data while moving.
- The recording epoch rate should be either one or five seconds.
- Single frequency receivers may be used although dual frequency receivers are preferred.

There are generally four parts of an RTK survey:

1. System check
2. Site calibrations
3. Corner measurements
4. Property corner stakeout

1. RTK System Check

This check consists of re-observing at least 3 known positions or all control points around the project. It is a measurement from the RTK base setup to another survey project control station or previously observed survey measurement point. The resulting observed position is then compared by inverse to the previously observed position for the known point. This inverse should be within the duplicate point tolerance, which is relevant to the project specifications.

When observing these measurements, the rover will be set for a duplicate point tolerance that meets the project specifications for the measurement of a known point with an existing point name. Manufacturers recommend that an RTK system check may also be made at any time during the course of each RTK survey session or at any time the base receiver(s) and rover receiver(s) are setup and initialized.

An RTK system check is designed to check the following:

- The correct reference base station is occupied.
- The GPS antenna height is correctly measured and entered at the base and rover. The recommended way to minimize ***blunders*** in measure-up is to record both meters and feet and make a comparison or to use a fixed height tripod/range pole.
- The receiver antennas are plumb over the station at the base and rover.
- The base coordinates are in the correct datum and the ***plane projections*** are correct.
- The reference base stations or the remote stations have not been disturbed.
- The radio-communication link is working.
- The RTK system is correctly initialized.
- ***Root mean square (RMS)*** values are within the manufacturer's limits.

2. RTK Site Calibration for Local Coordinate Systems

A calibration is the defining of a local coordinate system. A calibration or site registration is needed to relate World Geodetic System 1984 (WGS-84) positions to a local ***grid coordinate*** projection[7]. To do this, the user needs to be cautious about the datum epoch with which they are trying to work. Local North- East (NE) systems require one or more points that have coordinates in both WGS-84 and the chosen local grid coordinate projection system. A calibration is needed whenever points are to be staked out. The accuracy of points to be staked out will depend on the calibration accuracy. The accuracy and consistency of these points and of the GPS measurements to them will affect the quality of the calibration.

The number of points that can be used in a calibration is manufacturer and software dependent, although, it is recommended that the project be surrounded by control. Calibration points should be well distributed around the project exterior.

At least one, but preferably four or more points with existing chosen vertical datum and GPS measurements are required to introduce the vertical component into the project. These points should be distributed around the project in a "box" configuration such that they include any measured points within their boundary. The same points may be used for both horizontal and vertical calibration.

Care should be taken to keep relative equal spacing on control points. Clusters of control points should be avoided since the calibration software may place greater weight on these groups and skew or distort results.

A calibration, horizontal or vertical, may be used for subsequent projects as long as the new project falls within the calibrated area. Projects may be calibrated within a data collector or by using network adjustment software, then uploaded to the data collector (recommended for large projects).

7 This is in contrast to ***Universal Transverse Mercator (UTM)*** and ***State Plane coordinates***, which can be directly derived from WGS-84 using ***library projections***.

The calibration computation summary should be examined for reasonable results in the horizontal scale, maximum vertical adjustment inclination, and the maximum horizontal and vertical residuals. The RTK System Check can be used to verify the accuracy of the calibration.

3. RTK Corner Measurement

Corner measurements are usually made with RTK using one or more base configurations and one or more rover receiver configurations. RTK corner measurements shall only be made after completion of the system setup-check procedures.

When using RTK for corner measurements, follow the manufacturer's recommendations for observation times to achieve the highest level of accuracy, (for example, 180 seconds of time or when the horizontal (0.020 m) and vertical (0.050 m) precision has been met for a kinematic control point). Under optimal conditions, (clear sky, low ***positional dilution of precision (PDOP)***) a deviation from the manufactures suggested time could be appropriate. However, observation times should be set to account for field conditions, measurement methods, (i.e. topographic point or kinematic control point) and the type of measurement checks being performed. Observation time may need to be increased (i.e. 200+ sec) to obtain enough data if the field data is to be post-processed as a check.

Recommended methods for RTK corner measurement: RTK measurement method choices are based upon availability of control, terrain, timing, and staff and are optional.

- Observe the unknown point per guidelines described above and store the measurement (M1) in the data collector.
- Initialization may be checked at this time by forcing a loss of satellite lock. The best way to accomplish this is to ***invert the antenna***. The initialization can then be checked by a stakeout to the point or a second measurement (M2).
- RTK measurement verification: The baseline measurement (M1) to the found corner location or temporary point can be verified by the following method.

    Perform a check measurement (M2) from the same or different survey project control station used for the M1 measurement, at least 1 hour after the M1 measurement was taken. This resultant (M2) measurement can then be merged into the first (M1) measurement or stored separately (M2) for later analysis. The base antennas' height must be altered to insure an independent baseline.
- Note: The ***survey measurement tolerance*** of 4.6 cm is the maximum acceptable distance for M1 – M2 inverse. It should be accepted only under extremely poor GPS conditions due to tree cover, multi-path, etc. If this tolerance is exceeded, then one should measure with an alternate method or move to a different location.

RTK positioning is similar to a PPK or a total station radial survey. RTK does not require post-processing of the data to obtain a position solution. This allows for real-time surveying in the field. This method allows the surveyor to stake out similar to total station/data collector methods. RTK positioning is used for the survey measurement portion of a land survey.

Real-time surveying technology may utilize dual-frequency (L1/L2) techniques for initialization. The subsequent RTK measurements are then accomplished using the L1 carrier phase frequency and are subject to the baseline length limitations of 10 km. Other RTK measurement technologies can use the L1/L2 frequencies coupled with improved atmospheric models. This refined data is then transmitted from a central processor through a data link to determine baseline solutions allowing longer baseline measurement using RTK.

Network RTK surveys can be performed using multiple base stations that can be permanently installed or mobile. One system requires a set of fixed reference stations, which continuously transmit to a central computer server. The server performs sophisticated error checks. The server then models corrections for a rover based upon its location within the network of base stations from a position transmitted to the server via a two-way connection. This same connection then sends the standard corrections to the rover. Other systems operate using a series of independent base stations transmitting corrections that require the user to analyze the data from each station sequentially.

Caution: Steps must be taken for reobservation and to remove the antennae from any multipath areas. There is a need for redundancy during this type of work. This type of surveying is not acceptable for control since there is not enough redundancy or ability to analyze the data. The surveyor must make an informed decision when choosing the appropriate methodology to be used in a particular project area. For survey projects in a forest canopy environment with marginal sky visibility, static or fast-static GPS methods or even conventional optical methods should be considered in-lieu of using RTK or PPK.

- Field survey operations should be performed using the manufacturer's recommended receiver settings and observation times. Operations under adverse conditions, such as under a forest canopy, may require longer observation times than specified by the manufacturer, moving the setup to an area of less obstruction, or waiting for a change in the ***constellation***.

- Fixed height or adjustable height antenna tripods/range poles can be used for all rover GPS observations. The elevation of an adjustable height antenna tripod/range pole should be regularly checked to make sure slippage has not occurred.

- All plumbing/centering equipment should be regularly checked for proper adjustment.

4. RTK Corner Stakeout

RTK technology allows the surveyor to stakeout or set a corner from a known position. The system check will detect blunders and the initialization quality of the survey. A system check must be done before staking-out.

> Note: Exercise caution when using grid coordinates for a stakeout. It is important to apply the appropriate corrections for ***convergence***, elevation, and distance in accordance with the ***Public Land Survey System (PLSS)*** rules found in the Bureau of Land Management's "Manual of Instructions for the Survey of the Public Lands of the United States, 1973".

An acceptable corner stakeout procedure is:

Note: Other variations are acceptable. Ensure repeatable measurements with redundancy and rotate plumb pole to balance level bubble error and pole misalignment.

Navigate to a calculated corner location (e.g. Corner 'A') using coordinates from your local control base station (e.g. point 100).

Make a measurement at a temporary point called M1; inverse between the M1 coordinate and the calculated Corner 'A' coordinate; move the distance and direction to the calculated Corner A location. Repeat these measurements (M2) and corrections until you are satisfied that you occupy the calculated Corner A position, set a temporary stake at the position, then measure this location (M3) and overwrite the previously stored temporary point.

Invert or cover the rover antenna to force loss of satellite lock and initialization. Move 50'-150' away and reinitialize in a different multipath environment. Move back to the temporary point and check the location (M4).

Inverse between the M3 and M4 coordinates. If the measured positions of M3 and M4 are within the duplicate point tolerance of the calculated Corner A position then the initializations and measurements are correct. Note, in this example, the M3 and M4 measurements are a short duration (i.e. 30 seconds). If the measurements do not check, then move 50'-150' in a different direction, reinitialize, and remeasure the temporary point again. If these measurements do not check again, a new measurement should be made with a different satellite constellation at least 2 hours later.

Set the monument at the true Corner 'A' location.

As a validation check on the monument position, repeat measurement after 2 hours to get a new constellation. When measurements are made using dual RTK base stations, a move and reinitialization should be followed as outlined above.

Take a measurement on the set monument (e.g. for 180 seconds) and store the position with a different name (e.g. Corner 'A-1'). Optionally, set the receiver or data collector to store data for subsequent post-processing.

SECTION D: DATA PROCESSING AND ANALYSIS

Introduction to Loop Closure

The GPS surveying techniques are capable of generating centimeter accuracy results if the carrier phase ambiguities are correctly identified and constrained during data processing. The results are generally presented as Cartesian coordinate differences, referenced to the World Geodetic System 1984 (WGS 84) coordinate datum. These coordinate differences, or vectors, represent the *three-dimensional* coordinate difference between the reference and rover receiver. In addition to Cartesian coordinates, the vectors can be presented in terms of east, north, and height differences. This is commonly performed using a local horizon plane projection. Regardless of the manner in which the vectors are presented, closures of connecting baselines can aid in the detection of erroneous measurements. In the same manner in which a traverse misclosure is computed, the three dimensional misclosure of GPS vectors can also be determined. GPS surveys are not performed to generate traverse measurement equivalents; therefore, surveyors use manually selected baselines to form loops of baselines. The closures can be performed using a calculator, however, some GPS surveying systems provide loop closure utilities with the data processing software. Intelligent use of loop closures can enable erroneous baselines to be identified.

Checking Baselines Observed in Multiple Sessions

In order for a loop closure to be performed, GPS baselines are required from more than one observation session. If only one session is used, the baselines are correlated[8] and loop closures will tend to always indicate excellent results. This is due to the correlation between the baselines rather than the quality of the baselines. When multiple sessions are observed, a number of strategies for detecting poor quality vectors can be adopted. Consider the following example (see figure 5) where several redundant baselines have been observed.

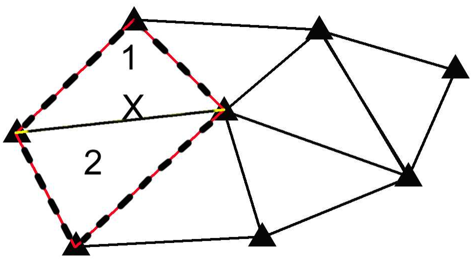


Figure 5

One strategy that may be adopted is to check each triangle; while trying to isolate any triangle, which reveals poor results. If each triangle is closed, it is likely that a bad baseline will affect more than one triangle. This technique results in often checking correlated baselines from the same session. It is also likely, however, that a session, which was too short to enable the ambiguities to be correctly resolved, will highlight two low quality baselines. Comparing all triangles will enable such instances to be detected if sufficient baselines are observed. In the example provided, if baseline X is erroneous, it can be anticipated that triangles 1 and 2 will highlight a poor closure. By performing a closure around the four-sided perimeter of triangles 1 and 2, the poor baseline can be highlighted. In addition, several of the points have been occupied on more than one occasion. Performing loop closures will aid in detecting whether antenna height errors are present in the data set.

8 Said of two or more observations (or derived quantities), which have at least one common source of error.
NOTE: See references in Appendix C for "Section 12 – GPS Surveying Survey Practice Handbook", Surveyors Board of Victoria, Australia

Example 4: Errors caused by lack of redundant measurements

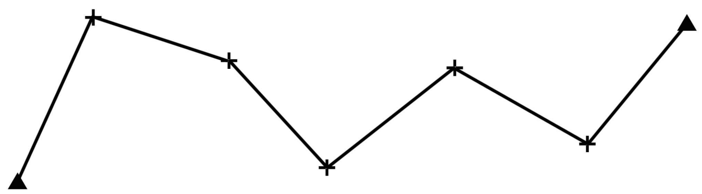
Experience has shown that during a static GPS survey (using dual-frequency geodetic equipment), it is possible to get good loop closure and have very small residuals in the network adjustment, yet still have a large errors in the adjusted positions of some stations. This can happen if stations are occupied in one session only.

In one situation, later traversing revealed an error of 2.4 meters in a station's coordinates that was occupied only once. Reprocessing the vectors in and out of the point, from a single session, changed the position by 2.4 meters to a position just a few hundredths different from one measured by ground traverse. Both the original and reprocessed GPS networks had good loop closure through the station in question. The vectors through the station in question had high variance numbers. There were compensating errors in the vectors in and out of the station. This can be attributed to the fact that both vectors into the station are computed from the same data set.

Example 5: Inability to find errors without redundant measurements

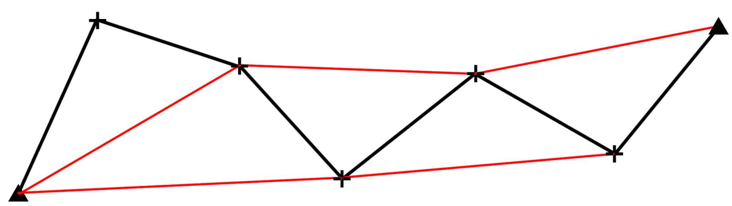
A GPS traverse was run between two monuments (see figure 6) about 34 km. apart. When we got to the second monument, we missed the published state plane coordinates by about 0.55 meters. This misclosure is within legal specifications since the distance is so far, but appears to be a concern. How can it be determined whether the error is within the traverse or one of the published coordinates?

Figure 6



ublished coordinates have an accuracy order, which is equated to a point error ± linear error, e.g. 8 mm ± 1: 1,000,000. One might expect the relative error between the example stations to be about 0.04 meters. The error found is likely the result of a poor quality GPS traverse, rather than bad control. New GPS measurements can be made between several other control stations to determine if one station is bad, but additional redundant measurements should also be made through the traverse stations, as demonstrated in figure 7. A series of braced quadrilaterals or triangles allow isolation of errors, as long as the measurements are all independent.

Figure 7



Internal Accuracy

Accuracy of a total station traverse can be computed based upon its misclosure. A loop of GPS baselines can be treated in a similar fashion: loop misclosure is computed in all three coordinate components and expressed as a ratio to the total distance of the loop. More commonly, however, this internal accuracy can be expressed in parts per million of the total baseline length. This will easily enable an assessment to be made regarding the loop closure performance in comparison to the manufacturer defined specifications, which are generally presented in parts per million. An example of a baseline closure is shown in figure 8. Six baselines with a combined length of more than eleven kilometers have been selected. The loops close to within a few centimeters, resulting in a part per million error of just over one and a half millimeters per kilometer.

Loop Closure

	Errors (m)	Errors (ppm)
X:	-0.0107	-0.9531
Y:	0.0043	0.3848
Z:	-0.0140	-1.2497

Total distance (m): 11,182.7581
Total error (ppm): 1.6181

Figure 8

Introduction to Network Adjustment

When performing network adjustments of GPS baselines, a least squares adjustment of the generated baselines is often performed once processing is complete and should follow the manufacturer's recommended procedures. These networks may comprise static and kinematic baselines. The network adjustment procedure has several functions in the GPS surveying process. The adjustment provides a single set of coordinates based on all the measurements acquired, as well as providing a mechanism by which baselines that have not been resolved to sufficient accuracy can be detected. A series of loop closures should be performed before the network adjustment procedure to eliminate erroneous baselines entering the adjustment process. A further feature of the network adjustment stage is that transformation parameters relating the GPS vectors to a local coordinate system can be estimated as part of the adjustment. The adjustment process can be done in several ways. The following sections highlight the major elements of the adjustment process.

Minimally Constrained Adjustment

Once the processed Cartesian vectors have been loaded into the adjustment module[9], an adjustment should be performed where one or no coordinates are constrained. The adjustment should be performed using the ***WGS84*** datum and appropriate estimates of station centering error. This solution provides a mechanism by which GPS baselines, which are not sufficiently accurate, can be detected. Once the ***minimally constrained adjustment*** has been performed, the surveyor should analyze the ***baseline residuals*** and statistical outputs (which will differ between adjustment programs) and ascertain whether any baselines should be removed from subsequent adjustments. This process relies on the baseline network being observed in such a manner to ensure that redundant baselines exist. Redundant baselines enable erroneous baselines to be detected as illustrated by Example 6 on page 37.

9 That portion of a GPS software application that is used to perform least squares adjustment of GPS networks.

Fully Constrained Adjustment

Once the minimally constrained adjustment has been performed and all unsatisfactory baseline solutions removed, a ***fully constrained adjustment*** can be performed. The constrained adjustment is performed to compute transformation parameters, if required, and yield coordinates of all unknown points in the desired coordinate system. The surveyor must ensure that sufficient points with known coordinates are occupied as part of the survey. The user should analyze the statistical output of the processor to ascertain the quality of the adjustment. Large residuals at this stage, after the minimally constrained adjustment has been performed, will indicate that the control points are non-homogeneous. It is, therefore, important that additional control points are occupied to ensure that such errors can be detected.

Error Ellipses

The ***standard deviation*** of the estimated coordinates is derived from the inverse of the normal matrix generated during formulation of the least squares process. Error ellipses for each point can be computed from the elements of this matrix. The ellipse presents a ***two standard deviation confidence region*** (95% certainty) in which the most probable solution based on the measurements will fall. Surveyors should base the quality of the adjustment process on the magnitude of these ellipses. Many contracts will specify the magnitude of error ellipses for both the minimally constrained and fully constrained adjustments as a method of prescribing required accuracy levels. The product documentation for the adjustment program will further indicate the manner in which the ellipse values are generated.

Independent Baselines (Non-Trivial Baselines)

For the least squares adjustment process to be successful, the surveyor must ensure that independent baselines have been observed. If more than one session is used to build the baseline network, then independent baselines will exist. In instances where one session is observed and all baselines adjusted, the ***measurement residuals*** will all be extremely small. This is due to the correlation that exists between the baseline solutions as they are derived from common data sets. This is not a problem as long as the surveyor is aware of the occurrence and does not assume that the baselines are of as high accuracy as implied from the network adjustment results. For each observing session, there are n-1 independent baselines where n is the number of receivers collecting data simultaneously, with measurements inter-connecting all receivers during a session. If the mathematical correlation between two or more simultaneously observed vectors in a session is not carried in the ***variance-covariance matrix***, the ***trivial baselines*** take on a bracing function simulating the effect of the ***proper correlation statistics***, but at the same time introducing a false redundancy in the count of the ***degrees of freedom***.

Error Analysis

The local accuracies of property corners are based upon the results of a least squares adjustment of the survey observations used to establish their positions. They can be computed from elements of a covariance matrix of the adjusted parameters, where the known

NSRS control coordinate values have been weighted using their one-sigma network accuracies. The covariance matrix can be found in the GPS software adjustment files, which may be labeled similar to *Station Coordinate Error Ellipses* within the *Error Propagation* tables.

Table 7: Error Propagation: An Example of Network Accuracy Analysis

Station Coordinate Error Ellipses (Meters) Confidence Region = 95%				
Station	Semi-Major Axis	Semi-Minor Axis	Azimuth of Major Axis	Elevation
4424	0.00000	0.00000	0°00’	0.00000
4427	0.00000	0.00000	0°00’	0.00000
4434	0.00000	0.00000	0°00’	0.00000
6369	0.00904	0.00850	2°10’	0.01222
6370	0.00913	0.00884	3°21’	0.01222
6371	0.00779	0.00753	3°37’	0.01045
Note: Points 4424,4427,and 4434 were held fixed as control				

Another useful table contains *Relative Error Ellipse* data and can be found near the other error tables (See Table 8: Relative Error Ellipses).

Table 8: Relative Error Ellipses: An Example of Local Accuracy Analysis

Relative Error Ellipses (meters) Confidence Region = 95%					
Stations From	Stations To	Semi-Major Axis	Semi-Minor Axis	Azimuth of Major Axis	Vertical
4424	6370	0.00913	0.00884	3°21’	0.01222
4424	6371	0.00779	0.00753	3°37’	0.01045
4427	6369	0.00904	0.00850	2°10’	0.01222
4427	6371	0.00779	0.00753	3°37’	0.01045
4434	6369	0.00904	0.00850	2°10’	0.01222
4434	6371	0.00779	0.00753	3°37’	0.01045
6369	6370	0.00863	0.00836	176°47’	0.01154
6369	6371	0.00820	0.00792	174°10’	0.01103
6370	6371	0.00757	0.00746	178°52’	0.01010

The 95% confidence circle representing a local accuracy can be derived from the major and ***minor semi-axis*** of the standard relative ellipse between two selected points. It is closely approximated from the major (a) and minor (b) semi-axis parameters of the standard ellipse and a set of coefficients. For circular error ellipses, the circle coincides with the ellipse. For elongated error ellipses, the radius of the circle will be slightly shorter than the major semi-axis of the ellipse. A rigorous treatment of this computation may be found in Appendix A but, for the purposes of this document, the radius of an error circle is equal to the major semi-axis of an associated error ellipse. The value of the largest error circle radius in the project should be adopted for reporting local project accuracy. Refer to Relative Accuracy and Local Accuracy in Section A of this document and several publications referenced in Appendix C for further study.

Data Integration

When integrating GPS measurements with conventional terrestrial survey measurements it is imperative that the surveyor verify:

- The type of GPS receivers and antennae used
- Geographic control coordinates have been properly transformed to State Plane values
- Appropriate value(s) for ***geoid separation*** is(are) adopted and applied for the survey
- The State Plane or local plane horizontal ***scale factors***, angular rotation factors, zone and units of measure are applied correctly.
- The ***elevation scale factor*** is applied correctly to determine ***ground distance*** at an average project elevation. A good check is to measure the distance between two known points with an EDM if the scale factor is significant.
- The appropriate datum has been used.
- There are no hidden or double transformations of the data.
- The computation and reporting of the basis of bearing is appropriate.
- Realistic standard error values for terrestrial obstructions, as well as for GPS observations.

SECTION E: PROJECT DOCUMENTATION

All GPS land surveying projects must be performed under responsible charge of a Professional Land Surveyor licensed to practice Land Surveying in Washington State (RCW 18.43.040). The Board of Registration for Professional Engineers and Land Surveyors considers the use of GPS surveying methods to establish geodetic control and make survey measurements to be the practice of Professional Land Surveying.

Proper documentation of a survey is important for future reconstruction of the measurements or presentation in court. Project documentation may be prepared and stored in the appropriate project file, either hard copy or electronic, by the surveyor in responsible charge as documentation of the GPS survey.

It is the surveyor's responsibility to ensure the appropriate safekeeping of survey observation and computation data. The resources allocated to archiving depend on the importance of the data and should be addressed on a case-by-case basis. The overriding principle is that, the more important the data, the more care should be taken in preserving it. When making an assessment the following items may be considered:

- Is the data of public interest
- Is the data of historic interest (the data may become useful for an apparently unrelated project)
- Is the data useful for future projects
- How much would it cost to replace the data
- How accessible should the data be (e.g. should hardcopy be scanned)
- In what format should the data be stored to make it easily used in the future (e.g. ***RINEX*** format for GPS data)
- On what medium should the information be stored (will the medium still be useable in the future, and will it require maintenance in the meantime)
- Should the data have redundant archiving
- Should there be off-site storage in case of a major catastrophe.

GPS data forms a record of the fieldwork and therefore should be archived so that it could be reprocessed or used as evidence in the future. Where possible, raw GPS observational data should be saved to Digital Versatile Disk (DVD), Compact Disk-Read Only Memory (CD-ROM), floppy disk, or tape. Data should be stored in the receiver native format or Receiver Independent Exchange (RINEX) format.

There are many GPS software packages available and although all return similar results, there is considerable variety in the information produced. Ideally, this would include all the mandatory and optional fields in the Receiver Independent Exchange (RINEX) file (ftp://igscb.jpl.nasa.gov/igscb/data/format/rinex210.txt), but at least the information below should considered as important:

- The station identifier
- The observing authority
- The ***antenna eccentricities*** (in all three dimensions) –measured to the ***Antenna Reference Point (ARP)***
- The receiver type
- The antenna type

- The GPS observables collected
- The satellites observed
- The observing times (start & finish)

Other key GPS solution elements that may be retained would include all elements specified in the ***Solution Independent Exchange (SINEX)*** format (ftp://igscb.jpl.nasa.gov/igscb/data/format/sinex.txt), but at least the information below should be considered:

- Processing Authority
- Software (name & version)
- Date & time of processing
- Unique solution identifier
- Data window
- Sampling interval
- Observables used
- Orbits used
- Solution options
- Ambiguity solution
- Station position(s)
- Variance-Covariance matrix
- Any additional comments

When Real-time kinematic is used, often raw GPS data is not logged, data collector files may therefore be retained for archive.

Narrative Report

A suggested surveyor's narrative report should include some combination of the following:

- The make and model of the GPS receiver, antenna, and related equipment.
- A summary of all field operations, including calibrations and duplicate point measurements.
- The baseline processing results and the software and version number used.
- The results of the Network adjustment including a summary of covariances, standard deviation or root mean square (RMS) values and the software and version number used.
- A network diagram, true line diagram or map showing the network configuration as designed.
- A list of the HARN, CORS, or reference stations used in the survey.
- A list of coordinates by station including the datum, geoid model, epoch, and measurement units used.
- Identification of local and/or network accuracies.
- Documentation of any variations from these guidelines.

Example 7 shows how a narrative report, including some items from the list above, can be added to a Record of Survey.

Example 7: Narrative Report for a Record of Survey[10]

Basis of Bearing

Washington Coordinate System, NAD83(1991), North Zone, Derived from GPS ties to the following control stations:

Station Designation	Northing	Easting
	U.S. Survey Feet	
GP37547 5	701,832.785	1,325,445.570
7215	678,676.536	1,324,359.222
7254	704,738.084	1,347,907.146

New control stations established during this survey using GPS methods

Station Designation	Northing	Easting
	U.S. Survey Feet	
7324	700,760.409	1,343,495.547
7325	697,903.937	1,343,110.240
7326	701,972.200	1,340,517.373

Convergence:

-00°55’33” @ Center ¼ of Section 31

Project Distance Factor

The areas and distances shown are grid. Divide the grid distance by 1.00000063 to compute the ground distance

Latitude Factor = 1.00003883
Elevation Factor = 0.9999618
Combined Factor = 1.00000063

Control Specifications and accuracy

Control for this survey is in compliance with the Federal Geodetic Control Committee’s “Geometric Geodetic Accuracy Standards and Specifications for Using GPS Relative Positioning Techniques, Version 5.0”, Reprinted with corrections on August 1, 1989, for Group C, Order 1 Surveys, field observations were made using four Trimble 5700 receivers, with zephyr geodetic antennae.

Corner ties were made by field traverse using a Leica TC1800L electronic theodolite. Traverses were either closed loops or began and ended at previously established control points. Traverse closures meet field traverse standards for Lambert grid traverses as specified in WAC 332-130-090. Corner monuments tied and monuments set were checked by double measure of both direction and distance.

Method of Survey:

This survey combined the use of four Trimble navigation 5700 receivers & Zephyr Geodetic antennae, a Topcon 603 total station, a Leica TC1800L electronic theodolite, and a steel tape. GPS control coordinates were obtained from a true least squares adjustment. Traverses are least squares or compass adjusted after a simple even distribution of angular error. Traverse closures meet or exceed field traverse standards for Lambert Grid Traverses as specified in WAC (Washington Administrative Code) 332-130-090.

10 That map to be filed with the County Auditor following direction found in Chapter 58.09 RCW, Surveys – Recording.

Reporting Bearings and Distances

Reporting a basis of bearing is required for a Record of Survey document. When using GPS technology, one may express the basis of bearing as a local bearing or assumed bearing in reference to a specific property line or adjacent record line. One may also express the basis of bearing as a "***Geodetic Bearing or Azimuth***" or reference it to the Washington Coordinate System Zone and datum. All ground distances may be determined at ground elevation, except where the requirements are for sea level, using the appropriate geoid model to determine the geoid separation. For a land survey, the height above the geoid and the orthometric height are considered the same.

Appendix A

GLOSSARY AND DEFINITIONS

1-sigma (1σ) find on pages 13, 39, 55, 57, 60, 61
Plus or minus 1 standard deviation. This value comprises 68.3% of the observations in a data set. The implication is that if an additional observation were taken, there is a 68.3% chance that it will fall within 1 standard deviation of the previously determined mean. It is computed by finding the square root of the sum of the squared residuals divided by the one less than the number of observations in the set.

1.96-sigma (1.96σ) find on pages 18, 28, 48, 57
Plus or minus 1.96 Standard Deviations. This value comprises 95% of the observations in a data set. The implication is that if an additional observation were taken, there is a 95% chance that it will fall within 1.96 Standard Deviations of the previously determined mean. See also *2-sigma*

2-sigma (2σ) find on pages 21, 61
Plus or minus 2 Standard Deviations. This value comprises 95.5% of the observations in a data set. The implication is that if an additional observation were taken, there is a 95.5% chance that it will fall within 2 Standard Deviations of the previously determined mean. See also *1.96-sigma*

3-sigma (3σ) find on page 56
Plus or minus 3 Standard Deviations. This value comprises 99.7% of the observations in a data set. The implication is that if an additional observation were taken, there is a 99.7% chance that it will fall within 3 Standard Deviations of the previously determined mean. An observation that falls outside of these limits is considered and anomaly or an outlier and should be rejected from the data set. See also *1-sigma and 2 sigma*

3-D Minimum-Constrained Least Squares Adjustment find on page 22
A least squares adjustment of a network of interconnected GPS measurements. Performed to test the internal consistency or "fit" of the measurements, this type of adjustment assumes that only one point in the network is "fixed". Used to find and eliminate erroneous or poor quality measurements from the network.

Accuracy find on pages 8, 9, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 25, 28, 29, 32, 33, 36, 37, 38, 39, 40, 41, 44, 46, 47, 48, 49, 52, 53, 54, 55, 56, 57, 58, 59, 60, 61, 63
The degree to which a measured quantity is related to the "true value". The closer a measurement is to the true value, the greater the accuracy of the measurement. How close a fix comes to the actual position.

Ambiguities find on pages 29, 30, 36, 50, 52, 56, 60
The initial bias in a carrier-phase observation of an arbitrary number of cycles. The initial phase measurement made when a GPS receiver first locks onto a GPS signal is ambiguous by an integer number of cycles because the receiver has no way of knowing the exact number of carrier wave cycles between the satellite and the antenna. This ambiguity, which remains constant as long as the receiver remains locked on the signal, is established when the carrier-phase data are processed.

Antenna Eccentricities find on page 42
The offset distance from the antenna phase center to the mark being measured.

Antenna Phase Center find on page 11
The electronic center of the antenna. It often does not correspond to the physical center of the antenna. The radio signal is measured at the APC. The phase center cannot be physically measured to, so the offset of the physical phase center from an external point on the antenna must be known and is commonly referenced to the Base/Bottom of Antenna.

ARP, Antenna Reference Point find on pages 42, 49
Determined by an actual external point that can be physically measured. Most commonly, this point will be the Bottom of Antenna (BA) and commonly referred to as the Bottom of Pre-amplifier. An example of the Antenna Reference Point is where the tribrach and antenna mount coincide.

Atmospheric Error find on page 11
The atmosphere contains electrically charged particles (ions). If the ionosphere is "excited" to a great extent (manifested in spectacular displays of the Northern Lights) GPS signals become deflected, thus inducing an error in GPS derived positions. Ionospheric delay is a bending or deflecting of the GPS radio signals as they pass through the Earth's ionosphere. Since the signals do not follow a direct path from satellite to antenna, they are delayed. This time delay messes up the computation of the pseudoranges from the satellites to the antenna, thus introducing an inaccuracy in GPS positions derived from them.

Base Station find on pages 24, 25, 31, 32, 34, 47, 49, 58
An antenna and receiver set-up on a known location. It is used for real-time kinematic or differential surveys. Data can be recorded at the base station for later post processing. The base station acts as the position from which all other unknown positions are derived.

Baseline Processing find on pages 9, 43, 57
The act of using a computer program to compute baseline solutions from satellite measurements. This may be done on a personal computer during post processing or within a receiver during real-time GPS surveying.

Baseline Residual find on page 38
Baseline or Vector: The difference in three-dimensional coordinates (X, Y, Z) computed from the difference in simultaneous carrier phase observations at two or more receivers. Residual: The difference between an estimated (adjusted) value and the observed value; specifically, $v = \mu_x - x$ where x is an observation of the estimated value μ_x.

Blunder find on pages 32, 34, 53, 55, 58
A mistake. An erroneous value resulting from a mistake. These have no place in a system of measurements as they produce an unwanted and unwarranted influence on the mean and other computed statistics, such as the Standard Deviation and Standard Error.

CA Code, Coarse Acquisition Code find on page 28
A family of PRN codes transmitted by GPS satellites. Each satellite is assigned one of 32 unique codes in the family. Each code consists of 1,023 chips and is sent at a rate of 1.023 megabits per second. The code sequence repeats every millisecond. The C/A-codes are Gold codes -- PRN codes that are distinguished by a very low cross correlation between any two codes (that is, they are nearly orthogonal). C/A-codes currently are transmitted only on the L1 frequency.

Carrier Phase find on pages 9, 20, 29, 30, 33, 36, 50, 53, 60
The accumulated phase of either the L1 or L2 carrier of a GPS signal, measured by a GPS receiver since locking onto the signal. Carrier phase is also called integrated Doppler.

Cartesian Coordinate find on pages 20, 36, 38
The Cartesian coordinate system is used by the WGS84 reference frame. In this coordinate system, the center of the system is at the earth's center of mass. The z-axis is coincident with the mean rotational axis of the earth and the x-axis passes through 0° N and 0° E. The y-axis is perpendicular to the plane of the x and z-axes.

CBN, Cooperative Base Network find on pages 56
A high-accuracy network of permanently marked control points spaced approximately 15-minutes by 15-minutes (25-50 km) apart throughout the United States and its territories. CBN contains additional stations as needed for safe aircraft navigation, or in areas of crustal motion. The National Geodetic Survey (NGS) responsibility for CBN is to assist and advise cooperating agencies in establishing spatial reference control according to accepted Federal standards and specifications.

Clock Error find on pages 11, 50
Since GPS measurements are based upon very precise time, small errors will affect position estimates.

Confidence Circle   find on pages 14, 17, 40

The 95% confidence circle representing a local accuracy can be derived from the major and minor semi-axes of the standard relative ellipse between two selected points. It is closely approximated from the major (a) and minor (b) semi-axis parameters of the standard ellipse and a set of coefficients. For circular error ellipses, the circle coincides with the ellipse. For elongated error ellipses, the radius of the circle will be slightly shorter than the major semi-axis of the ellipse. The radius r of the 95% confidence circle is approximated by:

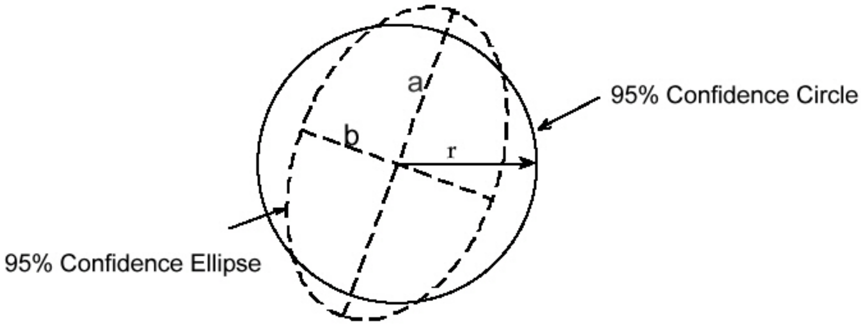


Figure 9

$r = K_p a$

Where

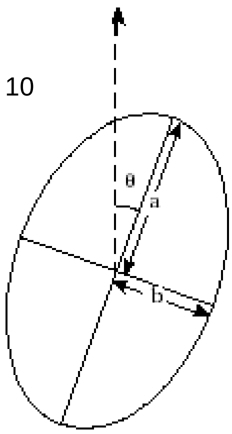
$K_p = 1.960790 + 0.004071 C + 0.114276 C^2 + 0.371625 C^3$

$C = b/a$

Note that the coefficients in the above expression are specific to the 95% confidence level, such that when the major semi-axis of the standard ellipse is multiplied by the value of K_p, the radius of the 95% confidence circle is obtained directly, and no further conversion is required (FGCS, 1995). Details on the circular confidence region may be found in Leenhouts (1985), the source of which has not been found.

Confidence Ellipse, 95%   find on pages 51, 54

Figure 10



The statistic used to represent the accuracy of the horizontal coordinates of a point. If the confidence ellipse represents network accuracy, then the accuracy of the point with respect to the defined reference system is represented. If the confidence ellipse represents a relative accuracy, then the accuracy of the point with respect to another adjacent point is represented, and may be used in conjunction with other relative accuracies at that point to compute its local accuracy.

The 95% confidence ellipse is centered on the estimated horizontal coordinates of the point as illustrated in Figure 10. The true position is unknown and can only be estimated through the measurements. The 95% confidence ellipse describes the uncertainty or random error in this estimated position, resulting from random errors in the measurements. There is a 95% probability that, in the absence of biases or other systematic errors, the true position will fall within the region bounded by the ellipse.

The 95% confidence ellipse for the horizontal coordinates of a point is derived from the covariance matrix of the estimated coordinates as computed using a least squares adjustment. It follows the actual distribution of the random error in the estimated position, and is the preferred means of representing accuracy when a detailed analysis of horizontal coordinate accuracy is required. See Error Ellipse

Constellation   find on pages 34, 35, 53, 56

A specific set of satellites used in calculating positions: three satellites for 2D fixes, four satellites for 3D fixes. All satellites visible to a GPS receiver at one time. The optimum constellation is the constellation with the lowest PDOP.

Constrained Adjustment   find on pages 23, 38, 39, 52

To hold (fix) a quantity (observation and coordinate) as true in a network adjustment.

Continuous Kinematic   find on pages 28, 30

Successive baseline solutions generated at every epoch of an unbroken observation set. Typically used to track a vehicle or platform in motion.

Convergence find on pages 34, 44
In this document, convergence is that angle between Washington State Plane Coordinate System grid and geodetic directions, also known as the mapping angle. A formula is used to determine convergence = geodetic direction – grid direction. For positions on the grid, West of the Central Meridian the convergence is a negative angle and East of the Central Meridian is a positive angle.

CORS, Continuously Operating Reference Station find on pages 13, 14, 15, 19, 31, 43, 55, 56, 57, 58
A GPS base station that may be part of the national network or of local origin. Data is often available over the Internet and may be available via radio or cell phone for real-time ***differencing***.

Covariance find on pages 12, 39, 40, 48, 43, 53, 61
The average value of the quantity x(r1) * x(r2); where x is a randomly varying function of the variable r, and r1 and r2 are two given values of r. Covariance describes the interdependence between variables and is typically expressed in a variance-covariance matrix where the diagonal elements are the variances of the corresponding variables, and those off the main diagonal are the covariance values.

Covariance Matrix find on pages 39, 40, 43, 48, 52, 61
A matrix that defines the variance and covariance of an observation. The elements of the diagonal are the variance and all elements on either side of the diagonal are the covariance.

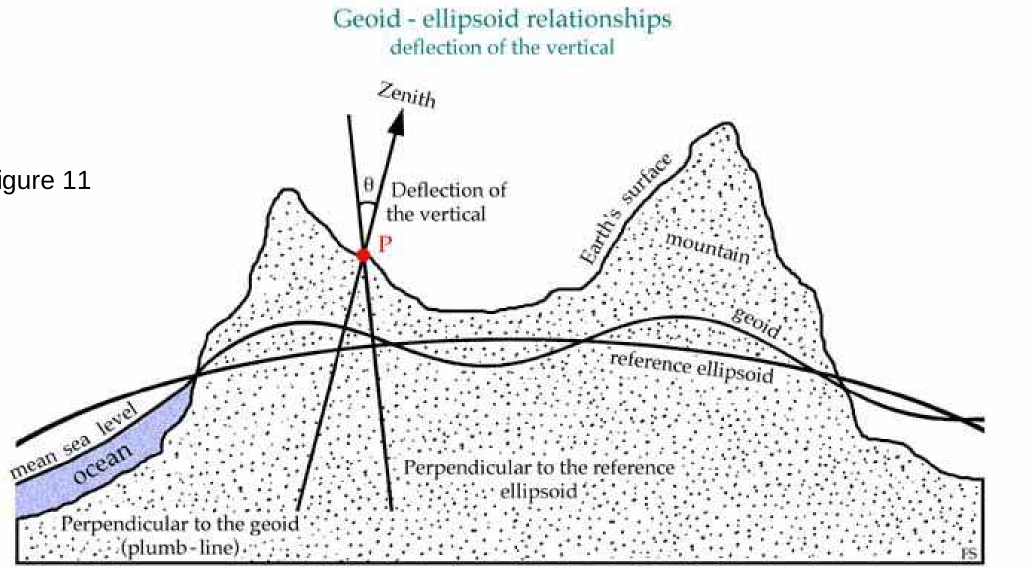
Cycles find on pages 46
The measurement interval used by a GPS receiver.

Cycle Slip find on pages 11, 25, 28
A discontinuity in GPS carrier-phase observations, usually of an integer number of cycles, caused by temporary signal loss. If a GPS receiver loses a signal temporarily, due to obstructions for example, when the signal is reacquired there may be a jump in the integer part of the carrier-phase measurement due to the receiver incorrectly predicting the elapsed number of cycles between signal loss and reacquisition.

Datum find on pages 13, 14, 19, 20, 22, 24, 32, 36, 38, 41, 43, 45, 51, 52, 55, 56, 61, 62
A frame of reference for coordinate systems that are used to locate or reference points of interest on a map. Some datums are designed to "fit" a specific portion of the Earth. For example, the North American Datum of 1927 was designed to "fit" the North American continent, but is not well suited for mapping other parts of the world. In contrast, the World Geodetic System of 1984 (WGS 84) datum and the closely related North American Datum of 1983 (NAD 83) can be applied worldwide and are compatible with global mapping systems such as the Global Positioning System.

Deflection of the Vertical find on page 21
The deflection of the vertical θ is at any point of observation the angle between the local vertical and the spheroidal normal.

Figure 11



Degrees of Freedom find on pages 39, 53
The number of redundant observations in a measurement system. For example, a triangle can be uniquely defined if you know the lengths of all three of its sides. If you were to measure the lengths of the three sides, and measure the three angles as well, you would have three more observations than would be minimally necessary to solve the problem. These three extra observations are thus redundant. You get one degree of freedom for each redundant observation in a system of measurements. The more redundant observations, the better. (See also, Mean)

DGPS, Differential Global Positioning System find on pages 28, 58, 59
A method for improving the accuracy of C/A code derived GPS positions. Raw C/A code positions are accurate to only about +/- 15 meters or so because not all of the delays in the radio signals broadcast by the GPS satellites (which are used to calculate pseudo ranges and positions) can be accounted for.

Differential correction works by modeling the delays in the signals, thereby improving the accuracy of pseudoranges derived from them. The delays can be estimated by collecting and analyzing GPS data that are collected at a point whose position is known beforehand. Thus, errors at the known point can be determined, and since the earth is such a large place, these errors (transformed into corrections) can be applied to data collected by other nearby GPS receivers.

Differencing or Phase difference processing find on pages 48, 53
Relative positioning computation of the relative difference in position between two points by the process of differencing simultaneous reconstructed carrier phase measurements at both sites. The technique allows cancellation of all errors, which are common to both observers, such as clock errors, orbit errors, and propagation delays. This cancellation effect provides for determination of the relative position with much greater precision than that to which a single position (pseudorange solution) can be determined.

DOP, Dilution of Precision find on pages 26, 33, 52, 55
A dimensionless number that accounts for the contribution of relative satellite geometry to errors in position determination. DOP has a multiplicative effect on the UERE[11]. Generally, the wider the spacing between the satellites being tracked by a GPS receiver, the smaller the position error. The most common quantification of DOP is through the position dilution of precision (PDOP) parameter. PDOP is the number that, when multiplied by the root mean square (rms) UERE, gives the rms position error. Other DOPs include the geometric dilution of precision (GDOP), horizontal dilution of precision (HDOP), and vertical dilution of precision (VDOP).

Double Difference find on pages 20, 24
The double difference observable involves linearly combining the measurements acquired to two satellites simultaneously measured by both receivers. In order to combine measurements in such a manner, the actual ***raw measurements*** are required by the processing module.

The double difference approach gives rise to the double difference ambiguity terms. One satellite is termed the reference satellite and is used during the development of all double differences. The solution reports and ***residual graphs*** generated by GPS processing packages portray the estimated double difference quantities. When the double differences are estimated as non-integer real numbers, the solution is said to be a ***float solution***. When the ambiguities are rounded to integers and constrained, the solution is fixed. The most accurate position results are generally provided by the fixed integer solution.

Dual-Frequency Receiver L1/L2 find on pages 28, 30, 31, 33, 37, 53
A type of receiver that uses both L1 and L2 signals from GPS satellites. A dual-frequency receiver can compute more precise position fixes over longer distances and under conditions that are more adverse because it compensates for Ionospheric delays.

Duplicate Point Tolerance find on pages 25, 31, 32, 35, 54
The maximum distance in an RTK system check (See Section B, II. D.1.) by which two observations of the same point can differ. It is also the maximum distance in an RTK survey that two observations of the same point can differ and still be recorded as the same point for least squares or multi-baseline analysis. The duplicate point tolerance for these guidelines is 2.5 cm.

Elevation Mask find on page 26
An angle, which is normally set to 15°. If you track satellites from above this angle, you usually avoid interference caused by buildings, trees, and multipath errors. Manufacturers recommend that you do not track satellites from below 13°, and the general requirements recommended by this document indicate the elevation mask should not be less than 15°.

Elevation Scale Factor find on page 41
A number by which a length measured on the ground is multiplied to obtain the geodetic length, or conversely.

11 UERE is **U**ser **E**stimated **R**ange **E**rror: the standard deviation of the errors in the psuedoranges of the satellites at the user's position

Ellipse find on pages 13, 39, 40, 41, 48, 51, 54, 55, 58, 59
A two-dimensional figure similar to a circle, but whereas a circle has but one radius, an ellipse has two radii of differing lengths. The longer of the two elliptical radii is known as the semi-major axis of the ellipse, the shorter as the semi-minor axis.

Ellipsoid find on pages 12, 20, 21, 22, 23, 52, 54, 59, 62, 65
A three-dimensional figure similar to a sphere, but whereas a sphere is based upon a circle, an ellipsoid is based upon an ellipse. Ellipsoids are used to approximate the size and shape of the Earth and in turn, form the basis for the various geodetic datums. In this case, semi-major and semi-minor axes' dimensions are chosen to provide a figure that "best fits" a particular region, e.g. North America, or the entire world.

Ellipsoid Height find on pages 12, 20, 21, 22, 23, 65
Most people understand elevation to mean height above mean sea level. However, GPS uses a mathematical model, called an ellipsoid, to represent the Earth. GPS receivers compute their 3D positions with respect to this model. Consequently, a GPS derived "elevation" (i.e., ellipsoid height) is a height above the ellipsoid model surface – not height above sea level. In Clallam County, Washington, a point's ellipsoid height will be about 20 meters (65 feet) less than the mean sea level elevation for the same point. This relationship between ellipsoid height and elevation varies worldwide.

Ephemeris
A description of the path of a celestial body indexed by time (from the Latin word, ephemeris, meaning diary). The navigation message from each GPS satellite includes a predicted ephemeris for the orbit of that satellite valid for the current hour. The ephemeris is repeated every 30 seconds and is in the form of a set of 16 Keplerian-like parameters with corrections that account for the perturbations to the orbit caused by the earth's gravitational field and other forces.

Epoch find on pages 19, 20, 29, 30, 31, 32, 43, 48, 51, 57, 59
A particular instant of time or a date for which values of data are given, or a given period of time during which a series of events take place.

Epoch Interval find on page 29
The measurement interval used by a GPS receiver; also called a cycle.

Equipotential Reference Surface find on pages 21, 52
A surface with the same potential (usually of gravity or of gravitation) at every point. This is also referred to as a level surface.

Error find on pages 9, 11, 12, 13, 14, 15, 16, 17, 18, 19, 21, 25, 29, 34, 36, 37, 38, 39, 40, 41, 44, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 57, 58, 59, 60, 64
The difference between a "true" value and a measurement. Since all measurements contain some error, it is impossible to determine a true value exactly, although it is possible to estimate how close the mean of a series of measurements is to the true value.

Error Circle find on pages 12, 13, 14, 15, 16, 18, 41
A variation on the error ellipse. The radius of an error circle is assumed equivalent to the semi-major axis of a particular error ellipse.

Error Ellipse find on pages 13, 39, 40, 41, 47, 48
A zone of uncertainty, elliptical in shape surrounding a survey point. At the center of the ellipse lies the most likely coordinate for the point, but statistically speaking, the point could lie anywhere within the computed error ellipse. In the case of GPS surveys, the dimensions of the ellipses are computed by combining the uncertainty in GPS vectors and set-up errors. Error ellipse's dimensions can be minimized by keeping ancillary equipment in good adjustment by performing redundant observations, and by collecting sufficient data to ensure that only high quality GPS vectors are included in the network. See Confidence Ellipse

Fast Static Method find on pages 11, 18, 19, 27, 30, 58, 64
A method of GPS surveying using occupations of up to 20 minutes to collect GPS raw data, then post processing to achieve sub-centimeter precisions. Typically, the occupation times vary based on the number of satellites (SVs) in view. 4 SVs take 20 minutes; 5 SVs take 15 minutes; 6 or more SVs take 8 minutes; all using a 15 second epoch rate.

FBN, Federal Base Network find on page 56
The Federal Base Network (FBN) is a nationwide network of permanently monumented stations spaced 100 kilometers apart with higher density in crustal motion areas. The FBN provides spatial reference control that has some of the most precise accuracies available today. The FBN is designed to provide the following local accuracies (with a 95% confidence level): 1 centimeter for latitudes and longitudes, 2 centimeters for ellipsoidal heights, 3 centimeters for orthometric heights, 50 microgals for gravity, 1 millimeter per year for crustal motion.

FGDC, Federal Geographic Data Committee   find on pages 12, 14, 19, 20, 64
The Federal Geographic Data Committee is a 19 member interagency committee composed of representatives from the Executive Office of the President, Cabinet-level and independent agencies. The FGDC is developing the National Spatial Data Infrastructure (NSDI) in cooperation with organizations from State, local and tribal governments, the academic community, and the private sector. The NSDI encompasses policies, standards, and procedures for organizations to cooperatively produce and share geographic data. http://www.fgdc.gov/

Fixed Solution   find on pages 20, 24
A solution obtained when the baseline processor is able to resolve the integer ambiguity search with enough confidence to select one set of integers over another. It is called a fixed solution because the ambiguities are all fixed from their estimated float values to their proper integer values.

Float Solution   find on pages 50
A solution obtained when the baseline processor is unable to resolve the integer ambiguity search with enough confidence to select one set of integers over another. It is called a float solution because the ambiguity includes a fractional part and is non-integer.

Fully Constrained Adjustment   find on page 39
A network adjustment in which all points in the network, which are part of a larger control network are held fixed to their published coordinate values. Used to merge smaller with larger control networks and old to newer networks.

GDOP, Geometric Dilution of Precision   find on pages 26, 50
The effects of the combined errors of four variables (latitude, longitude, altitude, and time) on the accuracy of a three-dimensional fix.

Geodetic Azimuth or Bearing   find on page 45
Geographic direction or pertaining to geodesy. Latitude and longitude readings are geodetic coordinates.

Geodetic Datum   find on pages 13, 50, 55
A mathematical model or reference frame upon which a map is based or to which surveyed positions are related e.g., WGS 84, NAD 83. Since the Earth is somewhat irregular in shape, different geodetic datums have been devised and are in use throughout the world. Positions that are expressed in one datum must be converted for use in another datum if spatial relationships are to be preserved and accurately presented.

Geoid   find on pages 21, 22, 23, 41, 43, 45, 55, 56
The undulating, but smooth, equipotential surface of the earth's gravity field, which coincides most closely with mean sea level. The geoid is the primary reference surface for heights.

Geoid Height   find on page 21
The distance between the geoid and ellipsoid, also known as geoid height.

Geoid Separation   find on pages 41, 45
The distance between the geoid and ellipsoid, also known as geoid height.

GIS, Geographic Information System   find on page 14
A computerized database and mapping application that allows for the integration, management, analysis, and display of data from a multitude of sources in a single program. GIS data generally have some kind of geographic component that make it possible to see spatial as well as mathematical relationships.

Grid Coordinate   find on pages 32, 34
A coordinate on a two-dimensional horizontal rectangular coordinate system, such as a map projection.

Ground Distance   find on pages 41, 44, 45
The horizontal distance measured at the mean elevation between two points.

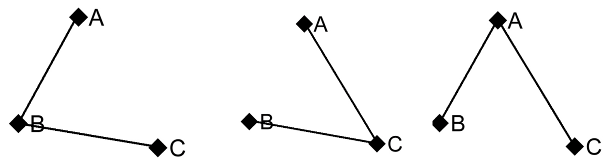
HARN, High Accuracy Reference Network   find on pages 14, 15, 19, 43, 55, 56, 57
The network of monuments in any state that are part of the Federal Base Network. See Federal Base Network.

Horizontal Residual
The difference between an estimated (adjusted) horizontal value and the observed horizontal value; specifically, $v = \mu_x - x$ where x is an observation of the estimated value μ_x.

HPGN, High Precision Geodetic Network   find on pages 19, 56, 57
The original name of the High Accuracy Reference Network.

Independent Baselines (Non-Trivial) find on pages 20, 24, 33, 39, 55, 61
For each observing session, there are n-1 independent baselines where n is the number of receivers collecting data simultaneously, with measurements inter-connecting all receivers during a session. Figure 12 is an illustration of all possible combinations of independent baselines that can be measured during a three-receiver session.

Figure 12



Non-trivial baselines are those vectors determined from differencing common phase measurements only once. Generally, independent baseline processors assume that there is no correlation between independent vectors. Trivial baselines may be included in the adjustment to make up for such a deficient statistical model. If the mathematical correlation between two or more simultaneously observed vectors in a session is not carried in the variance-covariance matrix, the trivial baselines take on a bracing function simulating the effect of the proper correlation statistics, but at the same time introducing a false redundancy in the count of the degrees of freedom. In this case, the number of trivial baselines in an adjustment is to be subtracted from the number of redundancies before the variance factor (variance of unit weight) is calculated. If this approach is not followed, trivial baselines are to be excluded from the network altogether.

Independent Occupations find on pages 8, 20, 24
An independent occupation is the set-up of the instrument over the station. These occupations are specified to help detect instrument and operator errors. Operator errors include those caused by antenna centering and height offset blunders. When a station is occupied during two or more sessions, back-to-back, the antenna pole or tripod should be reset and plumbed between sessions to meet the criteria for an independent occupation. A two-hour time break between sessions is very important to ensure constellation change.

Initialization or Initialized find on pages 28, 30, 32, 33, 34, 35, 57
Entering such data as time, time offset, approximate position, and antenna height into a receiver (cold start with no almanac) to help the unit find and track satellites.

Integer Number find on pages 46, 49
A whole number, e.g. 1, 2, 3, etc.

International Foot
0.3048 meters exactly.

Invert the Antenna find on pages 33, 35
Turn the GPS antenna away from the sky to loose satellite tracking and force reinitialization. This is a method of introducing redundancy into point measurement.

Ionosphere find on pages 11, 28, 46
That layer of the atmosphere approximately 30-300 miles above the earth's surface that contains electrically charged particles (ions).

Kinematic find on pages 11, 18, 19, 25, 26, 28, 30, 31, 33, 38, 43, 47, 48, 54, 57, 58, 59, 60, 64
Pertaining to motion or moving objects.

Kinematic Positioning find on page 30
Positioning a continuously moving platform by using GPS carrier-phase data while operating in a differential mode.

Kinematic Surveying find on page 30
A precise differential GPS surveying technique in which the roving user does not need to stop to collect precision information. Meter to centimeter-level accuracy is available using mode, dual-frequency, carrier-phase measurement techniques.

L-Band
The segment of the microwave portion of the radio spectrum nominally between 1 and 2 GHz.

L1 Carrier Phase Frequency find on pages 28, 33, 47, 48, 50, 56
See L1 frequency band.

L1 Frequency Band find on pages 28, 33, 47, 48
The 1575.42MHz GPS carrier frequency that contains the coarse acquisition (C/A) code and the encrypted P-code for the precise positioning service. Also carries the NAVDATA message used by commercial GPS receivers.

Land Survey find on pages 8, 9, 11, 18, 29, 30, 33, 42, 45, 55, 56
A survey, which creates, marks, defines, retraces, or reestablishes the location and extent of ownership boundaries or property lines. The term "practice of land surveying" within the meaning and intent of Chapter 18.43 RCW, shall mean assuming responsible charge of the surveying of land for the establishment of corners, lines, boundaries, and monuments, the laying out and subdivision of land, the defining and locating of corners, lines, boundaries and monuments of land after they have been established, the survey of land areas for the purpose of determining the topography thereof, the making of topographical delineations and the preparing of maps and accurate records thereof, when the proper performance of such services requires technical knowledge and skill.

Least Squares Adjustment find on pages 13, 18, 20, 22, 23, 30, 38, 39, 44, 46, 48, 50, 54, 55, 57, 60, 64
A method of distributing the effects of random errors in a system of measurements according to the laws of probability. Thus, least squares results in the most likely values. The simplest form of a least squares adjustment of a data set is the computation and use of the mean of the set. In a least squares adjustment, the sum of the squared residuals of a set of data is reduced to the minimum value possible.

Library Projection find on page 32
A projection with defined perameters

LiDAR, Light Detection and Ranging find on page 30
Light Detection And Ranging uses the same principle as RADAR. The lidar instrument transmits light out to a target. The transmitted light interacts with and is changed by the target. Some of this light is reflected / scattered back to the instrument where it is analyzed. The change in the properties of the light enables some property of the target to be determined. The time for the light to travel out to the target and back to the lidar is used to determine the range to the target.

Local Accuracy find on pages 13, 14, 18, 40, 41, 47, 48, 57
An average measure (e.g. mean, ***median***, etc.) of the relative accuracy of a point's coordinates with respect to other adjacent points at the 95% confidence level. For horizontal coordinate accuracy, the local accuracy is computed using an average of the radii of the 95% relative confidence ellipses between the point in question and other adjacent points. This indicates how accurately a point is positioned with respect to other directly connected, adjacent points in the local network. Based upon computed relative accuracies, local accuracy provides practical information for users conducting local surveys between control monuments of known position. Local accuracy is dependent upon the positioning method used to establish a point. If very precise instruments and techniques are used, the relative and local accuracies related to the point will be very good. This definition is taken from "Geospatial Positioning Accuracy Standards, Part 1: Standards for Geodetic Networks", by the Federal Geographic Data Committee; see References.

The reported local accuracy for land survey purposes will be computed from the error ellipses generated by a least squares or other multiple baseline statistical analysis, which is fully constrained to the survey project control.

Lock find on pages 28, 33, 35, 46, 47, 50, 57
The state in which a GPS receiver receives and recognizes a satellite's signals. If the signals are interrupted, the receiver experiences "loss of lock," a common cause of interruption in a kinematic or RTK survey.

Major Semi-Axis Parameter find on pages 41, 47
The radius of an ellipse in the longer of the two dimensions.

Manufacturer's Duplicate Point Tolerance find on pages 25, 31, 32, 35, 43
This is the manufacturer's specification for the amount of duplicate point tolerance (see duplicate point tolerance).

Manufacturer's Specifications find on pages 8, 18, 20, 24, 28
Equipment manufacturers provide a set of specifications within which their equipment will operate or recommendations for use of the equipment.

Mapping Projection find on page 19
Map projections often map the ellipsoid onto the plane by using a developable surface as intermediary. In a cylindrical map projection the ellipsoid is first mapped onto a cylinder; in a conical map projection the ellipsoid is first mapped onto a cone. Projections directly onto a plane are not designated as such, but are classified according to the location of the center of projection, location of the plane, etc.

Mean find on pages 18, 21, 30, 32, 42, 43, 46, 47, 48, 49, 50, 51, 52, 54, 56, 58, 59, 61, 62
The average of a series of measurements. The most likely value for a measurement that consists of multiple observations.

Measurement Residual find on page 39
An observation minus the mean of the set of observations. Residuals are used for computing Standard Deviation and Standard Error.

Measurement Uncertainty find on page 11
Arises from errors that naturally occur in a system of measurements, also know as random errors. Random errors are the only class of errors that remain in measurement system after blunders and systematic errors have been eliminated. Random errors' magnitudes and algebraic signs cannot be predicted with certainty. Nevertheless, their effects can be estimated. Random errors

in a measurement system do not accumulate as systematic errors do; instead, they propagate according to the square root of the sum of the squares.

Median find on page 54
In a data set, the median is the “middle value”. In a set of data, there are an equal number of values that are greater than and less than the median.

Meteorological Readings find on page 26
Usually in reference to various weather conditions that relate to GPS satellite signals passing through the atmosphere, e.g. temperature, barometric pressure, and humidity.

Minimally Constrained find on pages 38, 39
A network adjustment in which only enough constraints to define the coordinate system are employed. Used to measure internal consistency in observations.

Minor Semi-Axis Parameter find on pages 40, 47
The radius of an ellipse in the shorter of the two dimensions.

Multipath find on pages 11, 25, 28, 34, 35, 50
Interference, similar to ghosting on a television screen. Multipath occurs when GPS signals traverse different paths before arriving at the antenna. A signal that traverses a longer path yields a larger ***pseudorange estimate*** and increases the error. Reflections from structures near the antenna can cause the incidence of multiple paths to increase.

NAD83, North American Datum of 1983 find on pages 20, 49, 51, 61
Successor to NAD27, NAD83 can be applied worldwide and, since its origin point is coincident with the Earth’s center of mass, is directly compatible with global mapping systems such as the Global Positioning System.

NAD83 (1991), North American Datum of 1983(1991) find on pages 19, 23, 44
NAD83 network adjustment performed in 1991.

NAD83 (1998), North American Datum of 1983(1998) find on pages 19, 23
NAD83 network adjustment performed in 1998.

NAD83 (CORS), North American Datum of 1983(CORS) find on page 19
NAD83 positions determined at National CORS sites. See http://www.ngs.noaa.gov/CORS/metadata1/

NAD83 (NSRS), North American Datum of 1983(NSRS)
NAD83 network adjustment covering the entire United States as part of the National Spatial Reference System (NSRS) adjustment.

National Geoid Model find on page 22
The National Geodetic Survey publishes geoid model data covering many different regions, e.g. covering the continental U.S. (CONUS).

NAVD 88, North American Vertical Datum of 1988 find on pages 20, 22, 23, 56, 65
The vertical control datum used by the National Geodetic Survey for vertical control, adopted in 1988.

Network Accuracy find on pages 13, 14, 15, 19, 40, 48, 55
The absolute accuracy of the coordinates for a point at the 95% confidence level, with respect to the defined reference system. Network accuracy can be computed for any positioning project that is connected to the NSRS.

The network accuracy of a control point is a number that represents the uncertainty in the coordinates, at the 95% confidence level, of this control point with respect to the geodetic datum. For the NSRS network accuracy classification, the datum is expressed by the geodetic values at the NGS CORS. The local and network accuracy values at CORS sites are considered to be infinitesimal and to approach zero.

Network accuracy for land survey reporting purposes will be computed from the error ellipses generated in a least squares adjustment fully constrained to CORS or HARN stations or other GPS control stations which are referenced to the HARN or CORS as outlined in Section B: Types of GPS Surveying; Survey Project Control.

Non-Trivial Baseline find on pages 39, 52
See Independent Baseline.

NSRS, National Spatial Reference System find on pages 9, 13, 14, 19, 20, 24, 29, 39, 55, 59
A consistent national coordinate system that specifies latitude, longitude, height, scale, gravity, and orientation throughout the nation, as well as how these values change with time. The NSRS is defined and managed by the National Geodetic Survey (NGS). NSRS consists of the following components:

- The National CORS, a set of Global Positioning System Continuously Operating Reference Stations meeting NGS geodetic standards for installation, operation, and data distribution.

- A network of permanently marked points including the Federal Base Network (FBN), the Cooperative Base Network (CBN), and the ***User Densification Network (UDN).***
- A set of accurate models describing dynamic geophysical processes affecting spatial measurements.
- High Accuracy Reference Network (HARN) or High Precision Geodetic Network (HPGN) stations established by GPS observations.
- Vertical control marks, which are referenced to NGVD 29 or NAVD 88.
- All other horizontal and vertical marks established to define the NSRS.

NSRS provides a highly accurate, precise, and consistent geographic reference framework throughout the United States. Stations or marks established by GPS or high order classifications should be used with GPS survey projects.

One Sigma (1σ) find on pages 13, 39, 55, 57, 60, 61

Plus or minus 1 standard deviation. This value comprises 68.3% of the observations in a data set. The implication is that if an additional observation were taken, there is a 68.3% chance that it will fall within 1 standard deviation of the previously determined mean. It is computed by finding the square root of the sum of the squared residuals divided by the one less than the number of observations in the set.

Orthometric Height find on pages 12, 20, 21, 22, 23, 45, 51, 65

The height of a point above the geoid.

OTF, On-the-Fly find on page 28

GPS data is collected while the receiver is moving, e.g. locating an airplane during flight.

Outlier find on pages 22, 23, 25, 46

An observation that does not fit with other measurements in a survey network, or with the assumptions being made about the precision of the measuring system. An outlier falls more than 3-sigma standard deviators beyond expected limits for the observation. See also 3-simga

P-Code find on pages 28, 53, 54

A PRN code transmitted by GPS satellites. The code consists of about 2.353 1014 chips and is sent at a rate of 10.23 megabits per second. At this rate, it would take 266 days to transmit the complete code. Each satellite is assigned a unique one-week segment of the code that is reset at Saturday/Sunday midnight. The P-code is currently transmitted on both the L1 and L2 frequencies.

PDOP, Positional Dilution of Precision find on pages 33, 48, 50, 59

A measure of how the visible GPS constellation's geometry will affect the quality of GPS vectors. PDOP is used to select periods when the GPS satellites are most favorably positioned and thus optimal periods for collecting GPS data.

Phase Ambiguities find on pages 29, 30, 36

Integer bias term, Cycle ambiguity The unknown number of whole wavelengths of the carrier signal contained in an unbroken set of measurements from a single satellite at a single receiver.

Photogrammetric Survey

An indirect method for determining positions of features on the Earth's surface using aerial photographs and the methods of Photogrammetry.

Plane Projection find on pages 32, 36

A plane projection is an orderly system of lines on a plane representing a corresponding system of imaginary lines on an adopted terrestrial datum surface. Coordinates are published in many different coordinate systems using many different map projections, e.g., latitude-longitude, state plane, and Universal Transverse Mercator. Users must make certain that coordinate systems and map projections are used consistently in a GPS survey. See also, projection

PLSS, Public Land Survey System find on page 34

The Public Land Survey System (PLSS) is a rectangular survey system. It is called a rectangular system because wherever practicable the units are in rectangular form. The rectangular survey system divides land into townships and ranges. A regular township is six miles on a side bounded on the North and South by township lines, and the East and West by range lines. The township is divided into thirty-six sections, each one mile on a side, comprising about 640 acres, which was the basic unit under the Land Ordinance Act of 1785. No township or section is mathematically perfect for various reasons, including the fact that the earth's surface is not flat.

Point Standard Deviation find on page 13
The degree of certainty one has in the accuracy of a point's coordinates. Usually expressed as a plus-or-minus value in northing, easting, and elevation units, and usually at the 1-sigma level. Some software allows users to take points' standard deviations into account when performing a least squares network adjustment if they are to be used for project control.

Positional Accuracy find on pages 8, 11, 12, 14, 18, 28
The degree of certainty one has in the accuracy of a point's position. Usually expressed as a plus-or-minus value in units of the survey, as in, "...this point's horizontal position is accurate to +/- 0.05 feet." A point's positional accuracy can be related to its datum (network accuracy), or to the primary control points from which it is derived (local accuracy).

Positional Confidence find on page 11
A statement used to describe the level of confidence one has in a point's positional accuracy, as in, "…this point's horizontal position is accurate to +/- 0.05 feet at the 95% confidence level." See also, 1.96-sigma.

Positional Tolerance find on page 14
A method for analyzing the accuracy of survey point positions based on the precision of the measuring equipment and methodology used to determine the points' positions. Requires the use of the least squares adjustment method. An advantage to this analysis method is that it allows for integration of different data types, i.e. GPS and conventional, in the same survey network.

Post Processing find on pages 28, 47, 51
Applies to GPS vectors that are processed after the field observations have been completed. Involves downloading data files from the receivers to a computer equipped with GPS baseline processing software.

PPK, Post Processed Kinematic find on pages 25, 27, 30, 33, 34
Also known as "Stop-and-Go" kinematic. A radial GPS surveying technique wherein a base receiver is set up on a point whose position is known. A roving receiver is then "initialized" on the same or a different known point; after initialization, the rover occupies new points for a few epochs, and this data is recorded. Both the base and roving receivers must receive signals from the same set of at least four satellites, moreover, the rover cannot "loose lock" on the satellites if measurements are to continue uninterrupted. The technique differs from Real Time Kinematic in that the base and rover operate independently and are not in radio contact with one another. Processing of the kinematic GPS vectors is done in the office after field observations have been completed (hence, post-processed) and yields set baselines from the base receiver to each of the points occupied by the roving receiver. The technique is not suited to locations having sky obstructions and lacks redundancy. It is best suited to low accuracy mapping applications in open settings.

PPK-c, Post Processed Kinematic continuous find on page 25
This procedure is much like Post Processed Kinematic, but data is gathered continuously, without point identification. The data may be used to track an airplane during Photogrammetry operations or to map features while driving.

Precision find on pages 29, 31, 33, 49, 50, 51, 52, 53, 56, 58, 59, 60, 62
The degree of refinement of a measurement or set of measurements.

Primary Control Points find on page 13
The points to which all other points in a survey are related. Examples are published NGS horizontal control stations or benchmarks, HARN or HPGN stations; CORS stations or other points whose coordinates are known.

Projection, Mapping find on pages 19, 52, 54, 56, 59, 61
A mathematical model used for translating the positions of features on the Earth's surface, which is curved, to a flat surface, from which a map can be produced. Examples include the two-standard parallel Lambert conic projection used for the Washington Coordinate System, and the cylindrical Universal Transverse Mercator projection used for the UTM coordinate system.

Proper Correlation Statistics find on pages 39, 52
In a least squares adjustment, values that describe how well the observations fit the assumptions made about the precision of the tools used to make the observations. For example, in an adjustment that "passes the Chi-squared test" observations (GPS vectors) in the network fit together or "close" within limits that are consistent with assumptions made about centering errors and the like. There is a danger however: For example, unreasonably loose centering errors can be used to mask poor GPS vectors, yielding what are apparently good correlation statistics. Trivial vectors, when inadvertently included in a network adjustment can have the same effect, since the adjustment software will treat them as independent redundant observations, which they are not.

Pseudorange Estimate find on page 55
A pseudorange estimate is a range or distance from a GPS satellite to a GPS receiver that contains uncorrected or uncorrectable errors.

Public Record find on page 19
Generally considered a document, that is recorded at the office of the County Auditor. May also be a document with open access, e.g. from the Dept. of Natural Resources' Public Land Survey Office.

Radial Data Capture Techniques find on page 25
A GPS surveying technique wherein points' positions are developed from single GPS vectors from a base station to a roving receiver.

Random Error find on pages 9, 17, 48, 54, 55, 58
Errors that naturally occur in a system of measurements. Random errors are the only class of errors that remain in measurement system after blunders and systematic errors have been eliminated. Random errors' magnitudes and algebraic signs cannot be predicted with certainty. Nevertheless, their effects can be estimated. Random errors in a measurement system do not accumulate, as do systematic errors; instead, they propagate according to the square root of the sum of the squares.

Raw Measurements find on page 50
Unadjusted measurements.

RCW, Revised Code of Washington find on pages 7, 9, 13, 19, 20, 42, 44, 54, 61, 63
The Revised Code of Washington (RCW) is the compilation of all permanent laws now in force. It is a collection of Session Laws (enacted by the Legislature, and signed by the Governor, or enacted via the initiative process), arranged by topic, with amendments added and repealed laws removed. It does not include temporary laws such as appropriations acts. The RCW is published by the Statute Law Committee and is the official version of the code.

Reference Receiver find on page 30
A ground station at a known location used to derive differential corrections. The reference station receiver tracks all satellites in view, computes their pseudoranges, corrects these for errors, and then transmits the corrections to users.

Relative Accuracy find on pages 8, 13, 16, 41, 48, 54, 59, 60, 65, 66
The accuracy with which a user can measure position relative to that of another user on the same navigation system at the same time.

Relative Error Ellipse find on pages 13, 40
The uncertainty in distance and direction between two points, each of which have their own point error ellipse. Usually expressed in terms of bearing-plus-or-minus and distance-plus-or-minus. Also expressed as a parts-per-million or relative precision statement.

Residual find on pages 23, 33, 37, 38, 39, 46, 47, 50, 52, 54, 56, 58
The difference between an estimated (adjusted) value and the observed value; specifically, $v = \mu_x - x$ where x is an observation of the estimated value μ_x.

Residual Graph find on page 50
A graphic output from a computer program, which shows residual values. See Residual.

Resource Grade find on pages 18, 28
A type of GPS receiver with specifications limited in accuracy to 1-5 meters and suitable for measurement of many objects and features, but should not be used for property boundary surveys.

RINEX, Receiver Independent Exchange Format find on pages 42, 58, 65
A generic file format for GPS data. CORS data is available in this format. RINEX makes it possible to share data between different brands of GPS equipment.

RMS, Root Mean Square find on pages 32, 43, 50
An expression that describes the precision of the mean with respect to the data set from which it is derived, also known as the standard deviation of the set. Small standard deviations imply a data set with low scatter and hence, high precision.

Rover Receiver find on pages 29, 30, 31, 32, 33, 36
A GPS receiver that moves about (roves) collecting data at many different locations. Raw rover data is usually "corrected" using one of several techniques, such as real-time or post processed differential correction, real-time or post processed kinematic, or the fast static methods.

RTCM find on page 28
Radio Technical Commission for Maritime Services, Special Committee 104 has developed standard message types for use by differential GPS transmitting stations. The message content has been defined and hence when the RTCM-104 standard (version 2.2 is the latest) is implemented within a user receiver, it is able to decode and apply the DGPS corrections to its raw data in order to generate a DGPS-corrected coordinate.

RTK, Real-Time Kinematic find on pages 19, 24, 25, 27, 28, 31, 32, 33, 34, 35, 50, 59
The DGPS procedure whereby carrier-phase corrections are transmitted in real time from a reference receiver to the user's receiver. RTK is often used for the carrier-phase integer ambiguity resolution approach.

RTK-c, Real Time Kinematic continuous find on page 25
The DGPS procedure whereby carrier-phase corrections are transmitted in real time from a reference receiver to the user's receiver and the data is stored continuously at a given epoch rate. When continuous recording is being done, no point data is gathered since this procedure is typically used to track continuous movement of something like an airplane or vehicle. RTK is often used for the carrier-phase integer ambiguity resolution approach.

SA, Selective Availability find on page 11
Errors introduced into the GPS signal by the Dept. of Defense to limit the accuracy of GPS receivers available to civilians. Selective Availability has since been turned off increasing the accuracy of typically receivers from around 100 meters to around 15-25 meters.

Satellite Geometry find on pages 26, 29, 30, 31, 50
The relative positions of available GPS satellites at a given time, from the viewpoint of a GPS antenna. The set of positions that result in a high (or low) PDOP are often described as "poor (or good) satellite geometry.

Satellite Obstruction find on page 30
Any object that lies in the line-of-sight between the GPS antenna and the satellite.

Scale Factor find on pages 41, 50
A multiplier used on coordinate and other linear variables, such as for map projections and transformations.

Scale Parameters find on page 23
Multipliers used on coordinate and other linear variables, such as for map projections and transformation.

Semi-Axis find on pages 40, 41, 48, 54, 55
One-half of an ellipse's or ellipsoid's major (longer) or minor (shorter) axis. Analogous to radius in a circle or sphere.

SINEX, Solution Independent Exchange Format find on pages 43, 65
A solution output format recently developed by geodesists to permit the exchange of solution information between organizations, from which the original normal equation systems for precise GPS adjustments can be reconstructed. These reconstructed equation systems can be combined with other normal equation systems to create new GPS baseline solutions.

Standard Deviation find on pages 13, 39, 43, 46, 47, 54, 56, 57, 58, 61
An expression that describes the precision of the mean with respect to the data set from which it is derived. Small Standard Deviations imply a data set with low scatter and hence, high precision.

Standard Error Propagation find on page 60
Deals with the behavior of random errors. The accuracy of a measuring device can be estimated by computing its Standard Error, which is, in effect an estimate of the random errors that are likely to come from this source. However, random errors in a survey network do not simply accumulate since they can be positive or negative. Instead, they propagate according to the square root of the sum of the squares.

State Plane Coordinate find on pages 32, 37, 49, 56, 61
The plane-rectangular coordinate systems established by the United States National Geodetic Survey, one for each state in the United States, for use in defining positions of geodetic stations in terms of plane-rectangular (x and y) coordinates.

Static Method find on pages 29, 51, 58
See Static Positioning.

Static Positioning find on pages 29, 30
Location determination when the receiver's antenna is presumed to be stationary on the earth. In the case of pseudo-range-based techniques, this allows the use of various averaging techniques to improve the accuracy. Static Positioning is usually associated with GPS Surveying techniques, where the two GPS receivers are static for some observation period, which may range from minutes to hours (and even in the case of GPS geodesy, several days).

Standard Deviation find on pages 13, 39, 43, 46, 47, 54, 56, 57, 61
An expression that describes the precision of the mean with respect to the data set from which it is derived. Small Standard Deviations imply a data set with low scatter and hence, high precision. See one sigma error.

Station Error Ellipse find on page 13
The error ellipse at a GPS measured point. See Error Ellipse.

Stop & Go Kinematic   find on page 28
This is a GPS Surveying "high productivity" technique, which is used to determine centimeter accuracy baselines to static points, using site observation times of the order of 1 minute. Only carrier phase that has been converted into unambiguous "carrier pseudo-range" is used, necessitating that the ambiguities be resolved BEFORE the survey starts (and again at any time the satellite tracking is cut, e.g. due to signal obstructions). It is known as the "stop & go" technique because the coordinates of the receiver are only of interest when it is stationary (the "stop" part), but the receiver continues to function while it is being moved (the "go" part) from one stationary setup to the next. As the receiver must track the satellite signals at all times, hence the transport of the receiver from one static point to another must be done carefully.

Survey Measurement Tolerance   find on page 33
The maximum acceptable distance between two measured coordinates of the same point. This value is 4.6 cm. When measurements are made within this tolerance in an RTK survey, the two observations may be recorded as the same point number. These redundant measurements can then be included in a least squares or multi-baseline analysis. This worst-case condition should only be encountered in the most marginal field conditions for RTK surveys. These points should be noted in the surveyor's narrative report.

The survey measurement tolerance value of 4.6 cm is derived from standard error propagation relationships. It is based on the following formula; the square root of the sum of the squares of the survey measurement tolerance (4.6 cm) and the maximum allowable error of the survey project control (2cm) should approximately equal the maximum allowable error budget of the survey measurements (5cm). $\sqrt{4.6^2 + 2^2} = 5$

Survey Project Control   find on pages 18, 19, 20, 24, 25, 27, 29, 30, 31, 33, 54, 55
See Section B, Survey Project Control: Survey project control is a network of GPS stations tied to the NSRS or a local control scheme, which is surveyed to control all subsequent GPS survey measurements. Project control may also be used to bracket or surround a kinematic project with static points.

Survey Quality Statement   find on page 13
WAC 332-130-070 states that the accuracy or precision of field work may be determined and reported by either relative accuracy procedures or field traverse standards, provided the final result shall meet or exceed the standards contained in WAC 332-130-090. See also Section D, Narrative Report.

SV, Satellite Vehicle   find on pages 28, 51
Each satellite in the GPS fleet has a number designation, SV#, by which it is referred in status and condition reports. Data logging or viewing is also in reference to the specific SV#.

Systematic Error   find on pages 9, 48, 55, 58
Errors that can be computed and subsequently eliminated from a system of measurements. Example: Error in a chained distance caused by thermal expansion of the steel tape.

Terrestrial Survey   find on pages 12, 41
A survey on or of land.

Tidal Datum   find on page 20
For marine applications, a base elevation used as a reference from which to reckon heights or depths. It is called a tidal datum when defined in terms of a certain phase of the tide. Tidal datums are local datums and should not be extended into areas, which have differing hydrographic characteristics without substantiating measurements. In order that they may be recovered when needed, such datums are referenced to fixed points known as benchmarks.

Tide Station   find on page 20
A device that measures tide height using a recorder to send an audio signal down a half-inch-wide sounding tube and measure the time it takes the reflected signal to travel back from the water's surface. The sounding tube is mounted inside a 6-inch diameter protective well, which is similar to the old stilling well. In addition to measuring tidal heights more accurately, the system also records 11 different oceanographic and meteorological parameters. These include wind speed and direction, water current speed and direction, air and water temperature, and barometric pressure. Each tide station has a 7 digit reference number and can be found at the National Oceanic and Atmospheric Administration's site
http://tidesonline.nos.noaa.gov/monitor.html

Topographic Map Accuracy Standards   find on page 25
The vertical accuracy standard requires that the elevation of 90 percent of all points tested must be correct within half of the contour interval. On a map with a contour interval of 10 feet, the map must correctly show 90 percent of all points tested within 5 feet (1.5 meters) of the actual elevation.

Trivial Baseline   find on pages 39, 52
Trivial Baselines are those baselines formed when more than two GPS receivers are used simultaneously in the field to perform static GPS surveys. For example, when 3 receivers at points A, B, C are deployed only 2 baselines are independent (either A-B & A-C, AB & B-C, or AC & C-B), with the other one being trivial. This trivial baseline may be processed, but because the data used for this baseline has already been used to process the independent baselines, the baseline results should not be used for Network Adjustment or for quality control purposes unless the statistics (and variance-covariance matrix) are appropriately down weighted.

Tropospheric Error
Tropospheric Delay. Retardation of GPS signals caused by elements in the troposphere such as temperature, air pressure, and water vapor.

Two Sigma (2σ)   find on pages 21, 46
Plus or minus 2 Standard Deviations. This value comprises 95.5% of the observations in a data set. The implication is that if an additional observation were taken, there is a 95.5% chance that it will fall within 2 Standard Deviations of the previously determined mean. See also one sigma error

Two Standard Deviation Confidence Region   find on page 39
See Two Sigma.

UDN, User Densification Network   find on page 56
Comprises surveys that benefit the public and provide spatial reference for local infrastructure projects. UDN surveys must be connected by observations to Federal Base Network or Cooperative Base Network control points, in accordance with Federal Geodetic Control Subcommittee standards and specifications. When requested, NGS provides quality assurance, archiving, and distribution for UDN survey data.

U.S. Survey Foot
The United States uses a different foot for one activity. When the United States adopted the international yard in 1959, the U.S. Coast and Geodetic Survey, mappers of the nation, objected that converting all their geodetic data to international feet would be a horrendous undertaking. They were authorized to continue to use the previous definition of the foot, that of the Mendenhall order (U.S. Coast and Geodetic Survey Bulletin 26, April 5, 1893), one foot = $^{1200}/_{3937}$ meter. This foot is now known as the U.S. Survey foot = 1.000002 international feet, and is used only for land measurements.

UTM, Universal Transverse Mercator   find on pages 32, 57
A worldwide mapping system that uses rectangular coordinates to describe point's spatial relationships. There are 60 different UTM zones, each having unique defining parameters. Similar to state plane coordinate systems, but global in application. UTM mapping zones are based on cylindrical map projections whose axes are perpendicular to the Earth's axis of rotation.

Variance-Covariance Matrix   find on pages 39, 43, 48, 52
The set of numbers expressing the variances and covariances in a group of observations.

Vector   find on pages 20, 21, 22, 24, 28, 36, 37, 38, 39, 47, 51, 53, 56, 57
A line that has a direction and a magnitude. GPS Surveying is all about measuring vectors and combining them into closed figures or networks. GPS surveying instruments are used to measure vectors, and the vectors are used to compute points' coordinates.

Vector Baseline   find on pages 20, 21
A three-dimensional line between two points. The position of a point relative to another point. In GPS surveying, this is the position of one receiver relative to another. When the data from these two receivers is combined, the result is a baseline comprising a three-dimensional vector between the two stations.

WAAS, Wide Area Augmentation System   find on pages 28
A satellite navigation system designed by the Federal Aviation Administration (FAA) and created for aviation applications to boost the accuracy of G.P.S. satellite navigation. Each Wide Area Reference Station (WRS) provides correction data to a Wide Area Master Station (WMS), which computes a grid of correction data to be uplinked to a geostationary satellite (GEO). The geostationary satellite transmits the correction data (and also navigation data) to the user. Improvements in accuracy are approximated to be within 7 meters. WAAS signals are easily blocked in North America by terrain obstructions. Users may experience temporary loss of WAAS support, especially in wooded areas. Currently, WAAS is not fully implemented.

WAC, Washington Administrative Code   find on pages 9, 13, 15, 44, 60, 63
Regulations of executive branch agencies are issued by authority of statutes. Like legislation and the Constitution, regulations are a source of primary law in Washington State. The WAC codifies the regulations and arranges them by subject or agency.

Washington Coordinate System   find on pages 44, 45, 57, 63
Refer to Chapter 58.20 RCW. The system of plane coordinates under this chapter based on the North American datum of 1983 as determined by the National Geodetic Survey of the U.S. Dept. of Commerce.

Weighted Mean Average find on pages 18, 30
A method for computing the mean (or average) result of a data set that accounts for each observation's precision. Observations that are more precise are assigned more weight in the computation, thus they have more influence in the result than do observations that are less precise.

WGS 84, World Geodetic System 1984 find on pages 36, 38, 47, 49, 51
A mathematical model (or reference ellipsoid) of the Earth whose dimensions were chosen to provide a "best fit" with the Earth as a whole. WGS 84 and the closely related North American Datum of 1983 (NAD 83) are both Earth-centered and can be applied worldwide and are compatible with global mapping systems such as the Global Positioning System.

A set of parameters, established by the U.S. Defense Mapping Agency, for determining geometric and physical geodetic relationships on a global scale. The system includes a geocentric reference ellipsoid, a coordinate system, and a gravity field model. The ellipsoid is essentially that of the International Union of Geodesy and Geophysics Geodetic Reference System 1980. The coordinate system is a realization of the conventional terrestrial system, as established by the International Earth Rotation Service. The descriptions of the GPS satellite orbits in the navigation message are referenced to WGS 84.

Appendix B

WASHINGTON STATE LEGAL REFERENCES

Chapter 18.43 RCW –Engineers and Land Surveyors

18.43.020	Definitions
18.43.040	Registration Requirements

Chapter 58.20 RCW –Washington Coordinate System

58.20.110	Definitions
58.20.120	System designation – permitted uses.
58.20.130	Plane coordinates adopted – Zones.
58.20.140	Designation of system – Zones.
58.20.150	Designation of coordinates – “N” and “E”.
58.20.160	Tract in both zones – Description.
58.20.170	Zones – Technical definitions.
58.20.180	Recording coordinates – Control Stations.
58.20.190	Conversion of coordinates – Metric
58.20.200	Term – Limited use.
58.20.210	United States survey prevails – Conflict.
58.20.220	Real estate transactions – Exemption.
58.20.901	Severability – 1989 c 54.

Chapter 332-130 WAC –Minimum Standards for Land Boundary Surveys and Geodetic Control Surveys and Guidelines for the Preparation of Land Descriptions

WAC 332-130-020	Definitions.
WAC 332-130-060	Geodetic control survey standards.

For entire language for each law, visit http://slc.leg.wa.gov/

Appendix C

REFERENCES

"Analysis of Real-Time Kinematic and Fast Static/Kinematic Least squares Derived Coordinates Using a Wisconsin County UDN"; Paul Hartzheim and Darin Henkel, Wisconsin Department of Transportation, Trimble User Conference Proceedings, 1998

"Accuracy Standards for Positioning", Version 1.0 (DRAFT) July 1996, Natural Resources Canada-Geodetic Survey Division
http://www.geod.nrcan.gc.ca/index_e/products_e/publications_e/Accuracy_Standards.pdf

"Geometric Geodetic Accuracy Standards and Specifications for Using GPS Positioning Techniques", Version 5.0, May 11, 1988, reprinted with minor corrections August 1, 1989, Federal Geodetic Control Committee
http://www.ngs.noaa.gov/FGCS/tech_pub/GeomGeod.pdf

"Geospatial Positioning Accuracy Standards", FGDC-STD-007-1998, Federal Geographic Data Committee
Part 1: Reporting Methodology
http://www.fgdc.gov/standards/documents/standards/accuracy/chapter1.pdf
Part 2: Standards for Geodetic Networks
http://www.fgdc.gov/standards/documents/standards/accuracy/chapter2.pdf
Part 3: National Standard for Spatial Data Accuracy
http://www.fgdc.gov/standards/documents/standards/accuracy/chapter3.pdf

GPS World, GPS Reference Glossary, found at http://www.gpsworld.com/gpsworld/

"Manual of Instructions for the Survey of the Public Lands of the United States, 1973", Bureau of Land Management
http://www.az.blm.gov/cadastral/manual/manindex.html

"Preliminary Draft Guidelines for Geospatial Positioning Using GPS", Draft 7, June 10, 2001, Joe Evjen, National Geodetic Survey
http://www.ngs.noaa.gov/PROJECTS/GPSmanual/GPSguide.pdf

"New Mexico State Office Accuracy Standards for Land Surveys", New Mexico State Office, U.S. Bureau of Land Management

"Utilizing Real-Time Kinematic GPS for Control Surveys", Ronald Berg, Ministry of Transportation, Ontario, Trimble User Conference Proceedings, 1998

"Standards and Guidelines for Land Surveys using Global Positioning System Methods", Version 1.0 dated May 9, 2001, United States Department of Agriculture- Forest Service and United States Department of the Interior- Bureau of Land Management
http://www.blm.gov/nhp/efoia/wo/fy01/im2001-186attach1.pdf
http://www.ddl.org/figtree/pub/fig_2002/JS2/JS2_londe.pdf

“Surveyor’s Guidebook on Relative Accuracy” June 1995, Washington State Dept. of Natural Resources, Dept. of Natural Resources, Public Land Survey Office and Survey Advisory Board.
http://www.wa.gov/dnr/htdocs/plso/relacc.pdf

"Guidelines for Establishing GPS-derived Ellipsoid Heights (Standards: 2 cm and 5 cm)”, Version 4.3 dated November 1997, United States NOAA, NOS, National Geodetic Survey, NOAA Technical Memorandum NOS NGS-58
http://www.ngs.noaa.gov/pubs_lib/ngs-58.html

“NAVD 88 GPS-Derived Orthometric Heights”, Four Parts, dated 2002 and published by POB Online, by David B. Zilkoski
Part 1:
www.pobonline.com/CDA/ArticleInformation/features/BNP__Features__Item/0,2338,21678,00.html
Part 2:
cegis.ce.vt.edu/research/articles/ew%20articles/POBp46May01.pdf
Part 3:
cegis.ce.vt.edu/research/articles/new%20articles/POBp18Jul01.pdf
Part 4:
www.pobonline.com/CDA/ArticleInformation/features/BNP__Features__Item/0,2338,61910,00.html

“Standards and Practices for Control Surveys”, SP1 Version 1.5 dated May 2002, Australia, Inter-Governmental Committee on Surveying and Mapping (ICSM)
http://www.icsm.gov.au/icsm/publications/sp1/SP1v1-5.pdf

“Section 12 – GPS Surveying, Survey Practice Handbook”, Surveyors Board of Victoria, Dept. of Land Information, State of Victoria, Australia
http://www.land.vic.gov.au/web/root/domino/cm_da/lcnlc2.nsf/frameset/surveying then select Survey Practice Handbook link, then select GPS Surveying link.

“Surveying Using the Global Positioning System”, Surveyors Board of Victoria, Dept. of Land Information, State of Victoria, Australia,
http://www.surveyorsboard.vic.gov.au then select the document
http://www.surveyorsboard.vic.gov.au/documents/SurveyingusingGNSS.pdf

International standards for GPS files called RINEX & SINEX
ftp://igscb.jpl.nasa.gov/igscb/data/format/
ftp://igscb.jpl.nasa.gov/igscb/data/format/rinex210.txt
ftp://igscb.jpl.nasa.gov/igscb/data/format/sinex.txt

Barry, Austin B., *Errors in Practical Measurement in Surveying, Engineering and Technology*, John Wiley and Sons, Inc, reproduced by Landmark Enterprises Rancho Cordova, CA

Buckner, R.B., Ph.D. *Surveying Measurements and their Analysis*, Landmark Enterprises Rancho Cordova, CA

Mikhail, Edward M, Ph.D. and Gracie, Gordon, Ph.D.*Analysis and Adjustment of Survey Measurements* Van Nostrand Reinhold Co., New York, NY

Surveyor's Guidebook on Relative Accuracy Washington Department of Natural Resources, Olympia, WA, http://www.dnr.wa.gov/htdocs/plso/relacc.pdf

Definitions of Surveying and Associated Terms, Prepared by a joint committee of the American Congress on Surveying and Mapping and the American Society of Civil Engineers.
http://www.ghcc.msfc.nasa.gov/sparcle/sparcle_tutorial.html

Glossary Of GPS Terminology**,** Version 1998.23 http://www.rbf.com/cgcc/glossary.htm

GPS Glossary http://www.gps-com.gr/body_gps_glossary.html

Trimble *All about GPS*, http://www.trimble.com/gps/glossary.html

National Geodetic Survey, *Geodetic Glossary*

Satellite Navigation and Positioning Group, University of New South Wales, Sidney, Australia,
http://www.gmat.unsw.edu.au/snap/gps/glossary_r-z.htm

National Oceanic and Atmospheric Administration, Tidal Terminology Reference / Dictionary, http://www.baysail.com/tide_terms/