Chapter 8
Planning Data Collection with GPS

8-1. Scope

GPS surveying techniques are used to establish primary control and topographic feature mapping for USACE civil and military projects. Operational and procedural specifications for performing GPS surveys are a function of the accuracy required for a specific project or map product. To accomplish these surveys in the most efficient and cost-effective manner, and ensure that the required accuracy criteria are obtained, a detailed survey planning phase is essential. This chapter primarily focuses on survey design criteria (and related observing specifications) required to establish accurate horizontal and vertical control for USACE military construction and civil works projects. Planning considerations for mapping grade surveys using resource grade receivers are also covered; however, since these types of surveys are usually performed in near real-time or real-time, they require less mission planning.

8-2. General Planning Considerations for GPS Surveys

A number of factors need to be considered during the planning phase of a proposed GPS data collection survey. These include:

- Project Application--Purpose of Data Collection Survey
    - establishing primary control for subsequent location, topographic, hydrographic, or utility survey
    - general site plan, feature mapping, or GIS densification survey
    - number of horizontal points or benchmarks required or to be occupied
    - datum -- horizontal and vertical

- Accuracy Requirements
    - horizontal and vertical
    - will GPS provide the necessary accuracy

- Equipment Resources
    - In-house or contract
    - GPS receiver availability
    - other auxiliary equipment availability

- GPS Procedure
    - high accuracy--use centimeter-level static or kinematic carrier phase
    - medium accuracy--use meter-level code phase
    - low Accuracy--use 10-30 meter-level absolute positioning
    - RTK options for topographic mapping

- Network Design and Connections
    - static baseline connections to local project control
    - connections to NGRS/CORS points
    - code phase connections with wide-area commercial, USCG, or FAA WAAS networks

- Data Collection and Adjustment Techniques
  - feature, attribute, and format requirements
  - data collection session time
  - GPS initialization and calibration requirements
  - multiple/repeat baseline requirements
  - loop requirements
  - other quality control requirements
  - adjustment criteria and accuracy standards
  - metadata requirements
  - final survey report format

- Site Access and Restrictions
  - reconnaissance survey required
  - potential visibility restrictions or multipath problems

- Funding Considerations (impacts many of the above factors)

The above list is not exhaustive--numerous other project-specific conditions need to be considered. The following sections in this chapter attempt to address most of these planning considerations.

8-3. Project Control Function and Accuracy

The first step in planning GPS control surveys is to determine the ultimate accuracy requirements. Survey accuracy requirements are a direct function of specific project functional needs; that is, the basic requirements needed to support planning, engineering design, maintenance, operations, construction, or real estate. This is true regardless of whether GPS or conventional surveying methods are employed to establish project control. Most USACE military and civil works engineering/construction activities require relative accuracies (i.e. accuracies between adjacent control points) ranging from 1:1,000 to 1:50,000, depending on the nature and scope of the project. Few USACE projects demand relative positional accuracies higher than the 1:50,000 level (Second-Order, Class I). Since the advent of GPS survey technology, there has been a tendency to specify higher accuracies than necessary. Specifying higher accuracy levels than those minimally required for the project can unnecessarily increase project costs. Guidance on project accuracy requirements can be found in Table 8-1. This table provides recommended accuracies for features on various types of military and civil works projects. Similar guidance is also found in Part 4 of the *FGDC Geospatial Positioning Accuracy Standards* (FGDC 2002). Feature tolerances are abbreviated as metric (SI) or English inch-pound (IP) units.

Table 8-1. Recommended Accuracies and Tolerances for Engineering, Construction, and Facility Management

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Feature Position Tolerance Vertical SI/IP	Contour Interval SI/IP
DESIGN, CONSTRUCTION, OPERATION & MAINTENANCE OF MILITARY FACILITIES				
Maintenance and Repair (M&R)/Renovation of Existing Installation Structures, Roadways, Utilities, Etc				
General Construction Site Plans & Specs:	1:500	100 mm	50 mm	250 mm
Feature & Topographic Detail Plans	40 ft/in	0.1-0.5 ft	0.1-0.3 ft	1 ft
Surface/subsurface Utility Detail Design Plans	1:500	100 mm	50 mm	N/A
Elec, Mech, Sewer, Storm, etc	40 ft/in	0.2-0.5 ft	0.1-0.2 ft	
Field construction layout		0.1 ft	0.01-0.1 ft	
Building or Structure Design Drawings	1:500	25 mm	50 mm	250 mm
	40 ft/in	0.05-0.2 ft	0.1-0.3 ft	1 ft
Field construction layout		0.01 ft	0.01 ft	
Airfield Pavement Design Detail Drawings	1:500	25 mm	25 mm	250 mm
	40 ft/in	0.05-0.1 ft	0.05-0.1 ft	0.5-1 ft
Field construction layout		0.01 ft	0.01 ft	
Grading and Excavation Plans	1:500	250 mm	100 mm	500 mm
Roads, Drainage, Curb, Gutter etc.	30-100 ft/in	0.5-2 ft	0.2-1 ft	1-2 ft
Field construction layout		1 ft	0.1 ft	
Recreational Site Plans	1:1,000	500 mm	100 mm	500 mm
Golf courses, athletic fields, etc.	100 ft/in	1-2 ft	0.2-2 ft	2-5 ft
Training Sites, Ranges, and Cantonment Area Plans	1:2,500	500 mm	1,000 mm	500 mm
	100-200 ft/in	1-5 ft	1-5 ft	2 ft
General Location Maps for Master Planning	1:5,000	1,000 mm	1,000 mm	1,000 mm
AM/FM and GIS Features	100-400 ft/in	2-10 ft	1-10 ft	2-10 ft
Space Management Plans	1:250	50 mm	N/A	N/A
Interior Design/Layout	10-50 ft/in	0.05-1 ft		
As-Built Maps: Military Installation Surface/Subsurface Utilities (Fuel, Gas, Electricity, Communications, Cable, Storm Water, Sanitary, Water Supply, Treatment Facilities, Meters, etc.)	1:1000 or 50-100 ft/in (Army) 1:500 or 50 ft/in (USAF)	100 mm 0.2-1 ft	100 mm 0.2 ft	250 mm 1 ft
Housing Management GIS (Family Housing, Schools, Boundaries, and Other Installation Community Services)	1:5,000 100-400 ft/in	10,000 mm 10-15 ft	N/A	N/A
Environmental Mapping and Assessment Drawings/Plans/GIS	1:5,000 200-400 ft/in	10,000 mm 10-50 ft	N/A	N/A

Table 8-1. Recommended Accuracies and Tolerances for Engineering, Construction, and Facility Management (continued)

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Vertical SI/IP	Contour Interval SI/IP
Emergency Services Maps/GIS	1:10,000	25,000 mm	N/A	N/A
Military Police, Crime/Accident Locations, Post Security Zoning, etc.	400-2000 ft/in	50-100 ft		
Cultural, Social, Historical Plans/GIS	1:5000	10,000 mm	N/A	N/A
	400 ft/in	20-100 ft		
Runway Approach and Transition Zones:	1:2,500	2,500 mm	2,500 mm	1,000 mm
General Plans/Section	100-200 ft/in	5-10 ft	2-5 ft	5 ft
Approach maps	1:5,000 (H) 1:1,000 (V)			
Approach detail	1:5,000 (H) 1:250 (V)			
DESIGN, CONSTRUCTION, OPERATIONS AND MAINTENANCE OF CIVIL TRANSPORTATION & WATER RESOURCE PROJECTS				
Site Plans, Maps & Drawings for Design Studies, Reports, Memoranda, and Contract Plans and Specifications, Construction plans & payment				
General Planning and Feasibility Studies, Reconnaissance Reports	1:2,500 100-400 ft/in	1,000 mm 2-10 ft	500 mm 0.5-2 ft	1,000 mm 2-10 ft
Flood Control and Multipurpose Project Planning, Floodplain Mapping, Water Quality Analysis, and Flood Control Studies	1:5,000 400-1000 ft/in	10,000 mm 20-100 ft	100 mm 0.2-2 ft	1,000 mm 2-5 ft
Soil and Geological Classification Maps	1:5,000 400 ft/in	10,000 mm 20-100 ft	N/A	N/A
Land Cover Classification Maps	1:5,000 400-1,000 ft/in	10,000 mm 50-200 ft	N/A	N/A
Archeological or Structure Site Plans & Details (Including Non-topographic, Close Range, Photogrammetric Mapping)	1:10 0.5-10 ft/in	5 mm 0.01-0.5 ft	5 mm 0.01-0.5 ft	100 mm 0.1-1 ft
Cultural and Economic Resource Mapping Historic Preservation Projects	1:10,000 1000 ft/in	10,000 50-100 ft	N/A	N/A
Land Utilization GIS Classifications Regulatory Permit Locations	1:5,000 400-1000 ft/in	10,000 mm 50-100 ft	N/A	N/A
Socio-Economic GIS Classifications	1:10,000 1000 ft/in	20,000 mm 100 ft	N/A	N/A
Grading & Excavation Plans	1:1,000 100 ft/in	1,000 mm 0.5-2 ft	100 mm 0.2-1 ft	1,000 mm 1-5 ft
Flood Control Structure Clearing & Grading Plans (e.g., revetments)	1:5,000 100-400 ft/in	2,500 mm 2-10 ft	250 mm 0.5 ft	500 mm 1-2 ft

Table 8-1. Recommended Accuracies and Tolerances for Engineering, Construction, and Facility Management (continued)

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Feature Position Tolerance Vertical SI/IP	Contour Interval SI/IP
Federal Emergency Management Agency Flood Insurance Studies	1:5,000 400 ft/in	1,000 mm 20 ft	250 mm 0.5 ft	1,000 mm 4 ft
Locks, Dams, & Control Structures Detail Design Drawings	1:500 20-50 ft/in	25 mm 0.05-1 ft	10 mm 0.01-0.5 ft	250 mm 0.5-1 ft
Spillways & Concrete Channels Design Plans	1:1,000 50-100 ft/in	100 mm 0.1-2 ft	100 mm 0.2-2 ft	1,000 mm 1-5 ft
Levees and Groins: New Construction or Maintenance Design Drawings	1:1,000 100 ft/in	500 mm 1-2 ft	250 mm 0.5-1 ft	500 mm 1-2 ft
Construction In-Place Volume Measurement Granular cut/fill, dredging, etc.	1:1,000 40-100 ft/in	500 mm 0.5-2 ft	250 mm 0.5-1 ft	N/A
Beach Renourishment/Hurricane Protection Project Plans	1:1,000 100-200 ft/in	1,000 mm 2 ft	250 mm 0.5 ft	250 mm 1 ft
Project Condition Survey Reports				
Base Mapping for Plotting Hydrographic Surveys: line maps or aerial plans	1:2,500 200-1,000 ft/in	10,000 mm 5-50 ft	250 mm 0.5-1 ft	500 mm 1-2 ft
Dredging & Marine Construction Surveys				
New Construction Plans	1:1,000 100 ft/in	2,000 mm 6 ft	250 mm 1 ft	250 mm 1 ft
Maintenance Dredging Drawings	1:2500 200 ft/in	5,000 mm 15 ft	500 mm 2 ft	500 mm 2 ft
Hydrographic Project Condition Surveys	1:2500 200 ft/in	5,000 mm 16 ft	500 mm 2 ft	500 mm 2 ft
Hydrographic Reconnaissance Surveys	-	5,000 mm 15 ft	500 mm 2 ft	250 mm 2 ft
Offshore Geotechnical Investigations Core Borings /Probings/etc.	-	5,000 mm 5-15 ft	50 mm 0.1-0.5 ft	N/A

Table 8-1. Recommended Accuracies and Tolerances for Engineering, Construction, and Facility Management (continued)

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Feature Position Tolerance Vertical SI/IP	Contour Interval SI/IP
Structural Deformation Monitoring Studies/Surveys				
Reinforced Concrete Structures: Locks, Dams, Gates, Intake Structures, Tunnels, Penstocks, Spillways, Bridges	Large-scale vector movement diagrams or tabulations	10 mm 0.03 ft (long term)	2 mm 0.01 ft	N/A
Earth/Rock Fill Structures: Dams, Floodwalls	N/A	(same as above)	30 mm	15 mm
Levees, etc. -- slope/crest stability & alignment		0.1 ft (long term)	0.05 ft	
Crack/Joint & Deflection Measurements: piers/monoliths--precision micrometer	tabulations	0.2 mm 0.01 inch	N/A	N/A
<u>REAL ESTATE ACTIVITIES: ACQUISITION, DISPOSAL, MANAGEMENT, AUDIT</u> Maps, Plans, & Drawings Associated with Military and Civil Projects				
Tract Maps, Individual, Detailing Installation or Reservation Boundaries, Lots, Parcels, Adjoining Parcels, and Record Plats, Utilities, etc.	1:1,000 1:1,200 (Army) 50-400 ft/in	10 mm 0.05-2 ft	100 mm 0.1-2 ft	1,000 mm 1-5 ft
Condemnation Exhibit Maps	1:1,000 50-400 ft/in	10 mm 0.05-2 ft	100 mm 0.1-2 ft	1,000 mm 1-5 ft
Guide Taking Lines/Boundary Encroachment Maps: Fee and Easement Acquisition	1:500 20-100 ft/in	50 mm 0.1-1 ft	50 mm 0.1-1 ft	250 mm 1 ft
General Location or Planning Maps	1:24,000 2,000 ft/in	10,000 mm 50-100 ft	5,000 mm 5-10 ft	2,000 mm 5-10 ft
GIS or LIS Mapping, General Land Utilization and Management, Forestry Management, Mineral Acquisition	1:5,000 200-1,000 ft/in	10,000 mm 50-100 ft	N/A	N/A
Easement Areas and Easement Delineation Lines	1:1,000 100 ft/in	50 mm 0.1-0.5 ft	50 mm 0.1-0.5 ft	-

Table 8-1. Recommended Accuracies and Tolerances for Engineering, Construction, and Facility Management (concluded)

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Feature Position Tolerance Vertical SI/IP	Contour Interval SI/IP
HAZARDOUS, TOXIC, RADIOACTIVE WASTE (HTRW) SITE INVESTIGATION, MODELING, AND CLEANUP				
General Detailed Site Plans	1:500	100 mm	50 mm	100 mm
HTRW Sites, Asbestos, etc.	5-50 ft/in	0.2-1 ft	0.1-0.5 ft	0.5-1 ft
Surface Geotoxic Data Mapping and Modeling	1:500 20-100 ft/in	100 mm 1-5 ft	500 mm 1-2 ft	500 mm 1-2 ft
Contaminated Ground Water Plume Mapping/Modeling	1:500 20-100 ft/in	1,000 mm 2-10 ft	500 mm 1-5 ft	500 mm 1-2 ft
General HTRW Site Plans & Reconnaissance Mapping	1:2,500 50-400 ft/in	5,000 mm 2-20 ft	1,000 mm 2-20 ft	1,000 mm 2-5 ft

a. Project functional requirements. Project functional requirements must include planned and future design, construction, and mapping activities. Specific control density and accuracy are designed from these functional requirements.

(1) Density of control within a given project is determined from factors such as planned construction, site plan mapping scales, master plan mapping scale, and dredging and hydrographic survey positioning requirements.

(2) The relative accuracy for project control is also determined based on mapping scales, design/construction needs, type of project, etc., using guidance in Table 8-1 above. Most site plan mapping for design purposes is performed and evaluated relative to FGDC or American Society of Photogrammetry and Remote Sensing (ASPRS) standards--see references in Appendix A. These standards apply to photogrammetric mapping, total station mapping, and site plan mapping performed with GPS RTK techniques. Network control must be of sufficient relative accuracy to enable hired-labor or contracted survey forces to reliably connect their supplemental mapping work.

b. Minimum accuracy requirements. Project control surveys shall be planned, designed, and executed to achieve the minimum accuracy demanded by the project's functional requirements. In order to utilize USACE resources most efficiently, control surveys shall not be designed or performed to achieve accuracy levels that exceed the project requirements. For instance, if a Third-Order, Class I accuracy standard (1:10,000) is required for dredge/survey control on a navigation project, field survey criteria shall be designed to meet this minimum standard.

c. Achievable GPS accuracy. As stated previously, GPS survey methods are capable of providing significantly higher relative positional accuracies with only minimal field observations, as compared with conventional triangulation, trilateration, or EDM traverse. Although a GPS survey may be designed and performed to support lower accuracy project control requirements, the actual results could generally be several magnitudes better than the requirement. Although higher accuracy levels are relatively easy to achieve with GPS, it is important to consider the ultimate use of the control on the project in planning and designing GPS control networks. Thus, GPS survey adequacy evaluations should be based on the project accuracy standards, not those theoretically obtainable with GPS.

(1) For instance, an adjustment of a pair of GPS-established points may indicate a relative distance accuracy of 1:800,000 between them. These two points may be subsequently used to set a dredging baseline using 1:2,500 construction survey methods; and from 100-ft-spaced stations on this baseline, cross sections are projected using 1:500 to 1:1,000 relative accuracy methods (typical hydrographic surveys). Had the GPS-observed baseline been accurate only to 1:20,000, such a closure would still have easily met the project's functional requirements.

(2) Likewise, in topographic (site plan) mapping or photogrammetric mapping work, the difference between 1:20,000 and 1:800,000 relative accuracies is not perceptible at typical USACE mapping/construction scales (1:240 to 1:6,000), or ensuring supplemental compliance with ASPRS Standards. In all cases of planimetric and topographic mapping work, the primary control network shall be of sufficient accuracy such that ASPRS Standards can be met when site plan mapping data are derived from such points. For most large-scale military and civil mapping work performed by USACE, Third-Order relative accuracies are adequate to control planimetric and topographic features within the extent of a given sheet/map or construction site. On some projects covering large geographical areas (e.g., reservoirs, levee systems, installations), this Third-Order mapping control may need to be connected to/with a higher-order NGRS network to minimize scale distortions over longer reaches of the project.

(3) In densifying control for GIS databases, the functional accuracy of the GIS database must be kept in perspective with the survey control requirements. Performing 1:100,000 accuracy surveys for a GIS level containing 1-acre cell definitions would not be cost-effective; sufficient accuracy could be obtained by scaling relative coordinates from a US Geological Survey (USGS) quadrangle map.

d. Vertical accuracy. Establishing primary (i.e. monumented) vertical control benchmarks using carrier phase differential GPS methods requires considerable planning if traditional vertical accuracy standards are to be met. Since most Corps projects involve hydraulic flow of water in rivers, streams, pools, wetlands, etc., precise vertical control is essential within a project area; especially if construction is planned. Densification of vertical elevations with GPS requires sufficient control checks using conventional differential leveling, along with accurate geoid modeling. Therefore, an early evaluation needs to be made to determine if GPS-derived elevations will be of sufficient accuracy to meet project needs. Usually, a combination of GPS and conventional differential spirit leveling will be required. GPS standards and specifications needed to establish and densify vertical control network points are discussed in a later section of this chapter.

8-4. Selection of a GPS Survey Technique

Once a control project's accuracy requirement has been established, then the basic survey technique needed to meet this accuracy can be specified. The technique may or may not include GPS methods, or may require a combination of terrestrial (e.g., leveling and total station) methods and GPS. Depending on the accuracy requirements, either carrier phase or code phase GPS techniques may be selected. In general, monumented control points should be tied in using carrier phase methods--typically using static baseline connections to NGRS points. Some kinematic survey techniques (e.g., stop-and go, pseudo-kinematic) may also be employed to establish primary control when centimeter-level accuracy is required. External network connections and internal connections between monuments within the project area shall be performed using the network design guidance contained in the following sections of this chapter. Code phase techniques are more applicable to real-time, lower-order accuracy (meter-level) mapping projects, such as hydrographic surveying, GIS mapping, wetland delineation, shoreline delineation, etc.

8-5. Planning Differential Code Phase GPS Surveys

Meter-level accuracy code phase observations may be suitable for lower accuracy surveys, such as topographic feature mapping or real-time hydrographic surveys. Code phase observations should not be used for placing control on project monuments, nor should it be used for elevation measurement. Wide area differential code phase observations are easily achieved relative to NGRS stations that have supplemental communications to broadcast pseudorange corrections. These include FAA WAAS, USCG, and commercial systems. Since the distance from the broadcast points can significantly effect the ultimate positional accuracy, one should consult maps or web sites from the code phase provider to determine whether the distance is within acceptable limits (e.g., less than 150 miles for USCG sites and greater for systems that model over multiple stations).

a. DGPS providers. A real-time dynamic code-phase DGPS positioning system includes a reference station (master), communications link, and user (remote) equipment. If results are not required in real-time, the communications link could be eliminated and the positional information post-processed; however, such an operation is not practical for most construction support activities where immediate results are necessary. Since there are several DGPS services (USCG, FAA WAAS, and commercial subscription services) that provide real-time pseudorange corrections, it is recommended that these services be used before installing or using a local DGPS system. Only in circumstances where these services do not provide coverage should a local DGPS system be used.

b. Reference station. The reference station measures timing and ranging information broadcast by the satellites and computes and formats pseudorange corrections (PRC) for broadcast to the remote equipment. The reference station system typically consists of a GPS receiver, antenna, and processor. Using differential pseudoranging, the position of the user is found relative to the reference station. The pseudoranges are collected by the GPS receiver and transferred to the processor where pseudorange corrections are computed and formatted for data transmission. Many manufacturers have incorporated the processor within the GPS receiver, eliminating the need for an external processing device. The reference station processor computes the PRC and formats the corrections for the communications link to transmit to the remote unit or a survey vessel. The data transmission format is typically the Radio Technical Commission for Maritime Services Special Committee 104 (RTCM SC-104). The reference station processor may also be designed to perform quality assurance and integrity functions. This routine is required to determine the validity and quality of the computed PRCs. The reference station processor should be capable of computing and formatting PRC every 1 to 3 sec.

c. Reference station placement. The reference station is placed on a known survey monument in an area having an unobstructed view of the sky for at least 10 deg above the horizon. The antenna should not be located near objects that will cause multipath or interference. Areas with antennas, microwave towers, power lines, and reflective surfaces should be avoided.

d. Communications link. The communications link is used as a transfer media for the differential corrections. The main requirement of the communications link is that transmission be at a minimum rate of 200 bits per second (bps). Higher rates are required for wide-area networks. The type of communications system is dependent on the user's requirements.

e. Ultra high frequency (UHF) and very high frequency (VHF). Communications links operating at UHF and VHF are viable systems for the broadcast of DGPS corrections. VHF and UHF can extend out some 20 to 50 km, depending on local conditions. The disadvantages of UHF and VHF links are their limited range to line of sight and the effects of signal shadowing (from islands, structures, and buildings), multipath, and licensing issues.

f. Frequency authorization. Most RF communications links necessitate a reserved frequency for operation to avoid interference with other activities in the area. Transmitters with power outputs below 100 milliwatts (mW) do not require a frequency allocation and license for operation in the United States. Frequency authorization for the USACE must be obtained through the National Telecommunications and Information Administration (NTIA) of the US Department of Commerce for transmissions that exceed 100 mW. A district's frequency manager handles authorization and allocation of a frequency. No transmission can occur over a frequency until the frequency has been officially authorized for use. This procedure applies to all government agencies.

g. Satellite communications. There are several companies that sell satellite communications systems that can be used for the transmission of the PRCs. These systems are not as limited in range as a UHF/VHF system can be, but are usually higher in price.

h. User equipment. The user equipment is the most flexible facet of real-time code phase tracking DGPS. The remote receiver should be, at minimum, a multi-channel single-frequency (L1) C/A-code GPS receiver. The receiver must be able to accept the differential corrections from the communications link in the RTCM SC-104 format (see Chapter 7), and then apply those corrections to the measured pseudorange. The critical portion of the user equipment is the receiver update rate. Specific requirements will vary with different manufacturers and with the distance from the reference station. The output from the rover receiver should be in the NMEA 0183 format (see Chapter 7), because it is the most widely used standard for input into external devices, such as a hydrographic survey software package. For hydrographic applications, the user equipment also must be capable of maintaining positional tolerances for surveys at speeds of 7 to 10 knots. A DGPS receiver must not bias the position during vessel turns due to excess filtering.

i. Separation distances. In order to maintain meter-level positional accuracy tolerances, the maximum separation distance between a reference and remote station should generally not exceed 300 km, provided that differential tropospheric and ionospheric corrections are used. These corrections are not always applied to internal solutions on GPS receivers. The unaccounted tropospheric and ionospheric errors can contribute to horizontal position error on an average of 0.7 to 1.0 m per 100 km. A limiting factor of the separation distance is the type of data link used. If a DGPS is procured for hydrographic surveying, the reference station should be capable of being moved from one point to another. This will allow the user to move the reference station so that the minimum distance separation requirements are maintained.

j. Satellite geometry. In code phase DGPS, the Horizontal Dilution of Position (HDOP) is the critical geometrical component. The HDOP should be < 4 for most types of real-time meter-level positioning applications. The GPS constellation will maintain a HDOP of approximately 2 to 3 most of the time. In addition, quality control procedures need to be developed to ensure that systematic biases are not present in code phase positioning systems. These quality control procedures are spelled out in EM 1110-2-1003 (Hydrographic Surveying).

8-6. Field Reconnaissance for GPS Surveys

A good advance reconnaissance of all marks and features within the project is crucial to the expedient and successful completion of a GPS survey. The site reconnaissance should ideally be completed during the planning stage. The surveyor should also prepare a site sketch and brief description on how to reach the point since the individual performing the site reconnaissance may not be the surveyor that returns to occupy the known or unknown station.

a. Project sketch. A project sketch should be developed before any site reconnaissance is performed. The sketch should be on a 7-1/2-minute USGS quadrangle map, or other suitable drawing. Drawing the sketch on the map will assist the planner in determination of site selections and travel distances between stations.

b. Station descriptions and recovery notes. Station descriptions for all new monuments will be developed as the monumentation is performed. The format of these descriptions will follow that stated in EM 1110-1-1002 (Survey Markers and Monumentation). Station "Recovery Notes" should be written for existing NGRS network stations and Corps project control points, as detailed in EM 1110-1-1002. Estimated travel times to all stations should be included in the description. Include road travel time, walking time, and GPS receiver breakdown and setup time. These times can be estimated while performing the initial reconnaissance. A site sketch shall also be made on the description/recovery form. Examples of site reconnaissance reports are shown in Figures 8-1 and 8-2. A blank reconnaissance report form is included as Worksheet 8-1 (Figure 8-3), which may be used in lieu of a standard field survey book.

c. Waypoint reconnaissance navigation. Waypoint navigation is an option on most receivers, allowing the user to enter a geodetic position (usually latitude and longitude) of points or existing control monuments the user may wish to locate. The GPS antenna/receiver, fastened to a vehicle, range pole, or backpack, can then provide the user with real-time navigational information. The navigational information may include the distance and bearing to the point of destination (stored in the receiver), the estimated time to destination, and the speed and course of the user. The resultant message produced can then be used to guide the user to the point of interest. Waypoint navigation is an option that, besides providing navigation information, may be helpful in the recovery of control stations that do not have descriptions. If the user has the capability of real-time code phase positioning, the way point navigational accuracy can be in the range of 0.5 - 5 m.

d. Site obstruction/visibility sketches. The individual performing the site reconnaissance of a potential GPS point to be occupied should record the azimuth and vertical angle of all obstructions. The azimuths and vertical angles should be determined with a compass and inclinometer. Because obstructions such as trees and buildings cause the GPS signal transmitted from the GPS satellite to be blocked, the type of obstruction is also an important item to be recorded. The type of obstruction is also important to determine if multipath may be a problem. Buildings with reflective surfaces, chain-link fences, and antenna arrays are objects that may cause multipath. The site obstruction data are used to determine if the survey site is suitable for GPS observations. Obstruction data should be plotted on a "Station Visibility Diagram," such as that shown in Figure 8-4. A blank copy of this form is provided as Worksheet 8-2 (Figure 8-5). Ideally all GPS stations should have an unobstructed view 15 degrees above the horizon. Satellites below 10 degrees from the horizon should not be observed.

e. Suitability for kinematic observations. Clear, obstruction-free projects may be suitable for kinematic GPS surveys, as opposed to less-productive static methods. The use of kinematic observations will increase productivity by a factor of 5 to 10 over static methods, while still providing adequate accuracy levels. On many projects, a mixture between both static and kinematic GPS observations may prove to be most cost-effective.

f. Monumentation. All monumentation should follow the guidelines of EM 1110-1-1002 (Survey Markers and Monumentation).

g. On-site physical restrictions. The degree of difficulty in occupying points due to such factors as travel times, site access, multipath effects, and satellite visibility should be anticipated. The need for redundant observations, should reobservations be required, must also be considered.

h. Checks for disturbed existing control. Additional GPS baselines may need to be observed between existing NGRS or Corps project control points to verify their accuracy and/or stability.

i. Satellite visibility limitations. For most of CONUS, there are at least 4-5 satellites in view at all times--usually more. However, some areas my have less during times of satellite maintenance, unhealthy satellites, or DoD realignments for tactical purposes. Satellite visibility charts, available in most mission planning software, play a major part in optimizing network configurations and observation schedules.

j. Station intervisibility requirements. Project specifications may dictate station intervisibility for azimuth reference. This may constrain minimum station spacing.

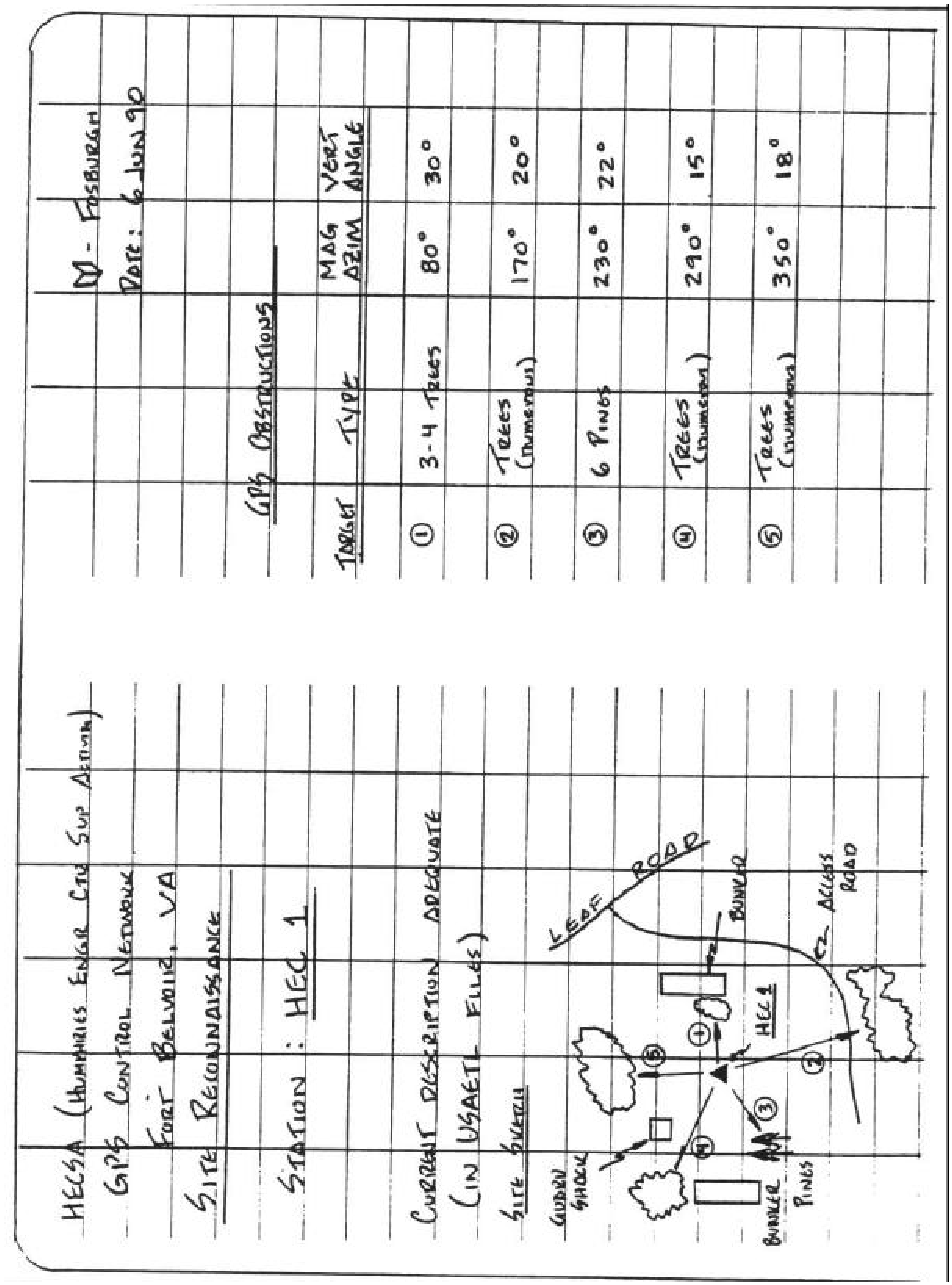

Figure 8-1. Field reconnaissance sketch in a standard field survey book format

SITE RECONNAISSANCE/REPORT ON CONDITION OF SURVEY MARK

Project for Which Reconnaissance was Performed DWORSHAK DAM

Station Name OROFINO Year Established 1933

State Code ID County POTTER Map Scale 1:24,000

Organization's Mark C & GS Map Sheet CLEARWATER

Search Performed By K. SMITH Date 4/12/89

Organization WALLA WALLA DISTRICT

Exact Stamping OROFINO 1933 Condition GOOD

Please report on the thoroughness of the search in case the mark was not recovered. Suggest changes in description, need for repairing or moving the mark, or other pertinent facts. Record letters and numbers found stamped in (not cast in) the mark.

THE MARK WAS RECOVERED USING THE 1970 DESCRIPTION. ADDITIONAL DESCRIPTIVED DATA:
THE MARK IS 89.7' W OF PP #6342, 62.4' NE OF AN 18" MAPLE, 42.0' S OF A 10" SPRUCE AND 2' E OF AN ORANGE WITNESS POST.
RECOVERED REFERENCE MARK OROFINO NO. 1 1933 GOOD
" " " OROFINO NO. 3 1970 GOOD

****

SKETCH

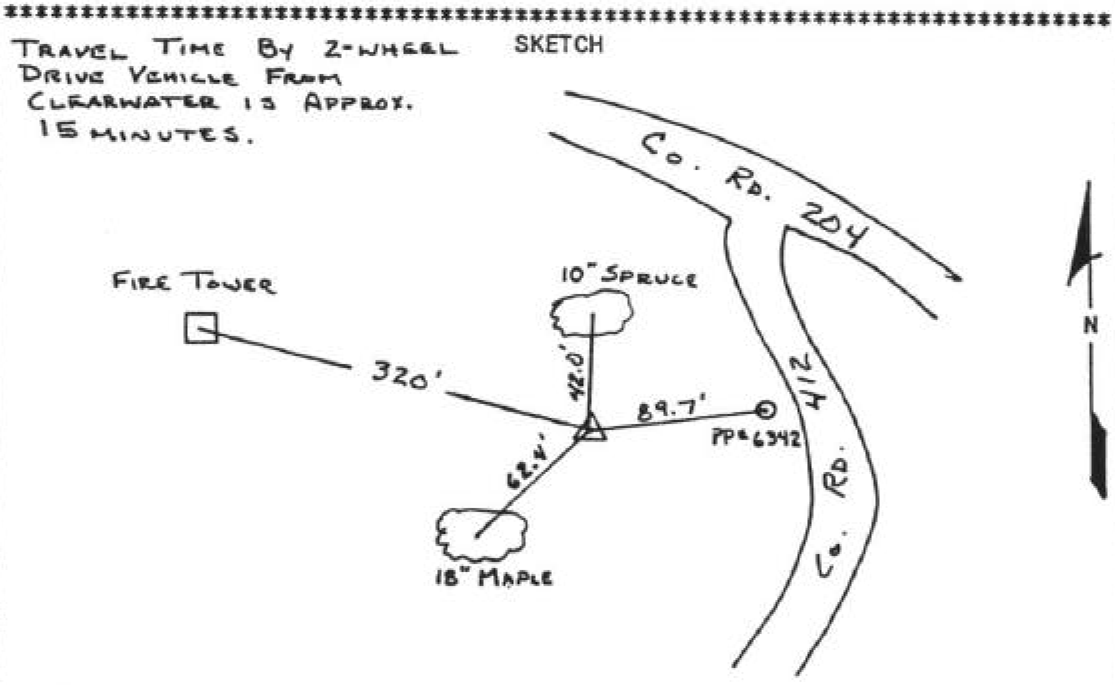

Figure 8-2. Reconnaissance survey sketch on notebook format

SITE RECONNAISSANCE/REPORT ON CONDITION OF SURVEY MARK

Project for Which Reconnaissance was Performed ____________________

Station Name ____________________ Year Established ________

State Code ____ County ____________________ Map Scale ____________

Organization's Mark ____________________ Map Sheet ____________

Search Performed By ____________________ Date ____________

Organization ____

Exact Stamping ____________________ Condition ____________

Please report on the thoroughness of the search in case the mark was not recovered. Suggest changes in description, need for repairing or moving the mark, or other pertinent facts. Record letters and numbers found stamped in (not cast in) the mark.

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SKETCH

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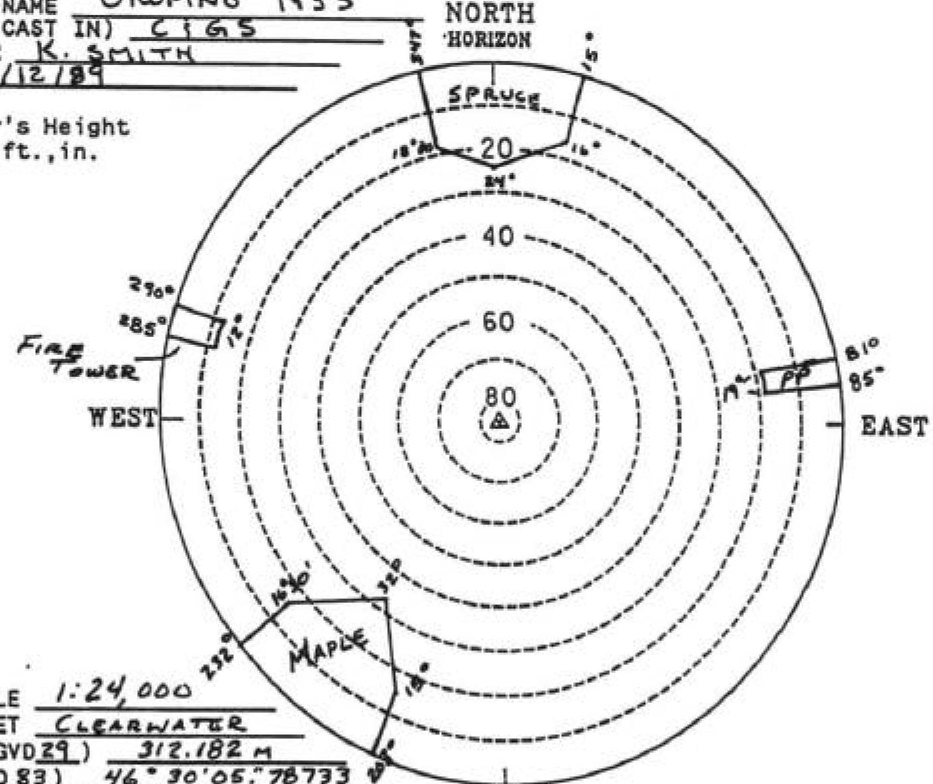
Figure 8-3. Worksheet 8-1, Site Reconnaissance Report

STATION VISIBILITY DIAGRAM

STATION NAME OROFINO 1933
AGENCY (CAST IN) C & GS
OBSERVER K. SMITH
DATE 4/12/89

Observer's Height
5' 10" ft., in.



MAP SCALE 1:24,000
MAP SHEET CLEARWATER
ELEV (NGVD 29) 312.182 m
LAT (NAD 83) 46° 30' 05." 78733 N
LONG (NAD 83) 116° 19' 27." 36405 W

Indicate the horizontal and vertical limits of all objects obstucting veiw of the sky exceeding 15 degrees above the horizon.

NO.	AZIMUTH	VERT ANGLE	DESCRIPTION	NO.	AZIMUTH	VERT ANGLE	DESCRIPTION
1	347°00'	18° 30'	Edge Spruce	11			
2	358°30'	24°00'	Peak Spruce	12			
3	15°00'	16°00'	Edge Spruce	13			
4	81°00'	19° 00'	N/Edge PP	14			
5	85°30'	19° 00'	S/Edge PP	15			
6	203°00'	15° 00'	Edge Maple	16			
7	215°30'	32° 00'	Peak Maple	17			
8	232°00'	16° 30'	Edge Maple	18			
9				19			
10				20			

Indicate distance, direction, frequency and power of known RF sources. Indicate distance, horizontal and vertical limits of any object in the vicinity that <u>may</u> cause radio signals to be reflected (multipath).

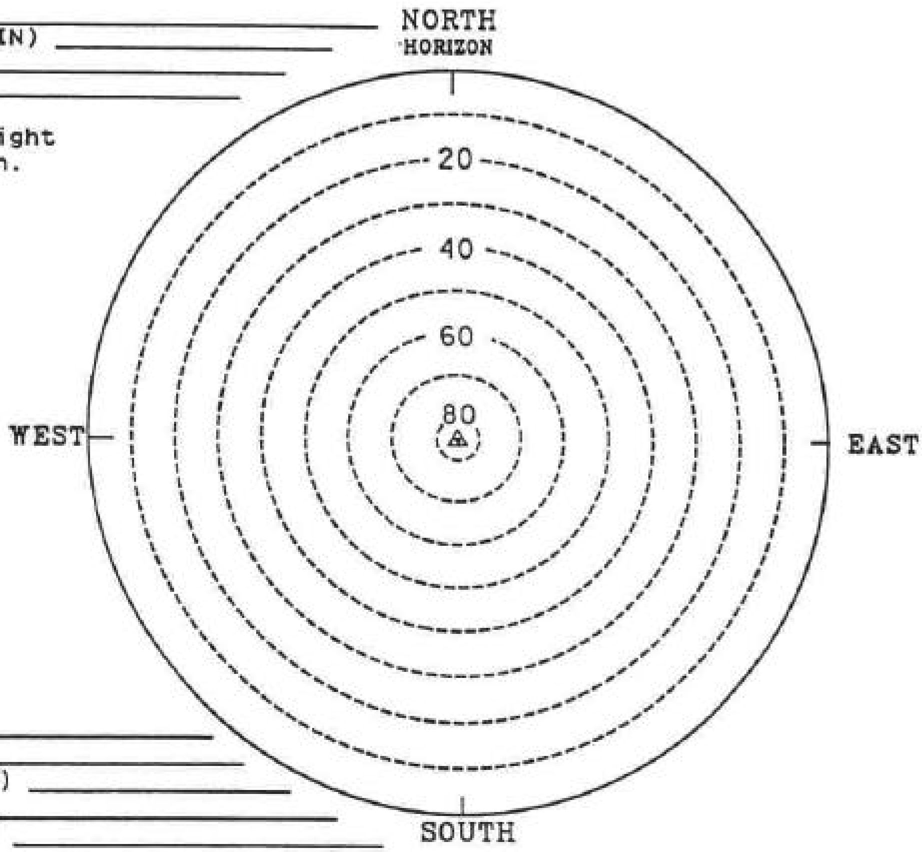
1. 320' 285° to 290° Vert ∠ 12° Fire Tower
2. (Due to Distance From Station & Low Elevation Angle, This is
3. Probably Not a Threat of Causing Multipath)

Figure 8-4. Typical example of a station visibility plot

STATION VISIBILITY DIAGRAM

STATION NAME ______________________
AGENCY (CAST IN) ____________________
OBSERVER ____________________
DATE ____________________

Observer's Height
_______ ft., in.



MAP SCALE ______________
MAP SHEET ______________
ELEV (NGVD___) ______________
LAT (NAD___) ______________
LONG (NAD___) ______________

Indicate the horizontal and vertical limits of all objects obstucting veiw of the sky exceeding 15 degrees above the horizon.

NO.	AZIMUTH	VERT ANGLE	DESCRIPTION	NO.	AZIMUTH	VERT ANGLE	DESCRIPTION
1				11			
2				12			
3				13			
4				14			
5				15			
6				16			
7				17			
8				18			
9				19			
10				20			

Indicate distance, direction, frequency, and power of known RF sources. Indicate distance, horizontal and vertical limits of any object in the vicinity that may cause radio signals to be reflected (multipath).

1. ______________________________
2. ______________________________
3. ______________________________

Figure 8-5. Worksheet 8-2, Station Visibility Diagram

8-7. Technical Criteria for Carrier Phase GPS Horizontal Control Surveys

Once a determination is made that high accuracy points are to be established on monumented project control, then, by default, carrier phase techniques are required. The National Geodetic Survey, in conjunction with the FGDC, has developed standards and specifications for performing static carrier phase DGPS surveys. One of these standards is universally recognized for performing DGPS surveys, namely:

> "*Geometric Geodetic Accuracy Standards and Specifications for using GPS Relative Positioning Techniques*," (Version 5.0 dated 11 May 1988, Corrected to 1 August 1989), published by the Federal Geodetic Control Committee (FGCC), which now falls under the Federal Geographic Data Committee (FGDC)

Although this standard was developed for static GPS methods in 1988 before a full GPS constellation was available, many of the recommended techniques and criteria are still valid. Table 8-2 below contains recommended USACE standards for precise horizontal GPS surveys. It applies to both static and some kinematic techniques. It is synopsized from the FGCC 1988 standard but addresses Corps-specific DGPS techniques and criteria that are more relaxed and practical than the rigid FGCC 1988 standards. In addition, many of the higher-order NGRS network densification criteria in FGCC 1988 are not applicable to Corps military and civil works project control surveys. The following sections provide guidance for some, but not all, of the factors to be considered in designing a GPS network and planning subsequent observing procedures. The FGCC 1988 standard contains a more detailed discussion of many of these network design factors--this standard should be thoroughly reviewed by those involved in such a process. More detailed explanations of the FGCC 1988 criteria can also be found in various GPS textbooks listed in Appendix A--e.g., Van Sickle 2001.

Table 8-2. Minimum Standards for Horizontal Control Survey Design, Geometry, Connection, and Observing Criteria--Carrier Phase Differential GPS Surveys

Classification Order						
Criterion		1st	2nd, I	2nd, II	3rd, I	3rd, II
Relative Accuracy	ppm	10	20	50	100	200
	1 part in	100k	50k	20k	10k	5k
General Criteria						
Required connections to existing NGRS network		Yes	Yes	W/F/P	W/F/P	W/F/P
Baseline observation check required over existing control		Yes	Yes	W/F/P	W/F/P	No
Number of connections with existing network (NGRS or Local)						
	Minimum	3	2	2	2	2
	Optimum	3+	3	3	3	3
	Vertical	[	see Table 8-4			]
Max distance from network to nearest point in project (km)		50	50	50	50	50
Maximum distance to nearest CORS point		[	no specified limit			]
Field Observing Criteria						
Repeat baseline occupations--not less than 10% or at least		2	2	2	2	2
Loop closure requirements:						
Maximum number of baselines/loop		10	10	20	20	20
Maximum loop length, km, not to exceed		100	100	200	N/R	N/R
Loop misclosure, ppm, not more than		10	20	50	100	200
Spur baseline observations:						
Number of sessions/baseline		2	2	2	2	2
Required antenna height measurement per session		2	2	2	2	2
Dual-frequency L1/L2 observations required:	< 20-km lines	Yes	No	No	No	No
	> 20-km lines	Yes	Yes	Yes	Yes	Yes
Recommended elevation mask above horizon, degrees		15	15	15	15	15

Notes:
1. Abbreviations used in this table are explained as follows:
W/F/P--Where feasible and practical
N/R--No requirement for this specification--usually indicates variance with provisional FGCC GPS specifications
2. Classification orders refer to intended survey precision for USACE application, not necessarily FGDC standards designed to support national network densification.

a. Project size. The extent of the project will affect the GPS survey network shape. Many civil works navigation and flood control projects are relatively narrow in lateral extent but may extend for many miles longitudinally. Alternatively, military installations or reservoir/ recreation projects may project equally in length and breadth. The optimum GPS survey design will vary considerably for these different conditions. For higher-order surveys, Table 8-2 recommends that the project area shall be surrounded by optimally 3 connections to NGRS control, or more if precise vertical densification is being performed. For projects extending along linear waterway systems, far more NGRS connections will be required. Given the high density of NGRS/CORS stations in CONUS, this requirement is easily achieved for most projects other than coastal navigation sites. See also FGCC 1988 for further discussion.

b. Required density of control. The spacing of new points on a civil or military installation project needs to be assessed for each control survey, based on the supplemental purposes of the control points (photogrammetric mapping control, GIS mapping control, construction, etc.). The type of GPS survey scheme used will depend on the number and spacing of points to be established, which again is a project-specific requirement. Often, a combination of static GPS, RTK GPS, and conventional survey densification will prove to be the most cost-effective approach.

c. Absolute GPS reference datums. Coordinate data for GPS baseline observations are referenced and reduced relative to WGS 84, an earth-centered (geocentric) coordinate system. As stated in Chapter 3, this system is not directly referenced to, but is closely related to (for all practical engineering survey purposes) GRS 80, upon which NAD 83 is related. GPS data reduction and adjustments are normally performed using the WGS 84 coordinate system (X-Y-Z), with baseline vector components ($\Delta X, \Delta Y, \Delta Z$) measured relative to this coordinate system. Although baseline vectors are measured relative to the WGS 84 system, for most USACE engineering and construction applications these data may be used in adjustments on NAD 27 (Clarke 1866 spheroid).

(1) If the external network being connected (and adjusted to) is the published NAD 83, the GPS baseline coordinates may be directly referenced on the GRS 80 ellipsoid since they are nearly equal. All supplemental control established is therefore referenced to the GRS 80/ NAD 83 coordinate system.

(2) If a GPS survey is connected to NAD 27 (State Plane Coordinate System of 1927, or SPCS 27) stations that were not adjusted to the NAD 83 datum, then these fixed points may be transformed to NAD 83 coordinates using high-level geodetic transformation routines, or approximately transformed using USACE program CORPSCON (see EM 1110-1-1004), and the baseline reductions and adjustment performed relative to the GRS 80 ellipsoid.

(3) Alternatively, GPS baseline connections to NAD 27 (SPCS 27) project control may be reduced and adjusted directly on that datum with resultant coordinates on the NAD 27. Refer to EM 1110-1-1004 regarding state plane coordinate transforms between SPCS 27 and SPCS 83 grids. Conversions of final adjusted points on the NAD 27 datum to NAD 83 may also be performed using CORPSCON. It is strongly recommended that all existing NAD 27 control be transformed to NAD 83 by direct baseline connection to NGRS points

(4) Ellipsoid heights " h " referenced to the GRS 80 ellipsoid differ significantly from the orthometric elevations " H " on NGVD 29, NAVD 88, or dynamic/hydraulic elevations on the IGLD 55-IGLD 85 systems. This difference (geoid separation, or " N ") can usually (but not always) be ignored for horizontal control surveys. This implies " N " is assumed to be zero and " $h = H$ " where the elevation may be measured, estimated, or scaled at the fixed point(s).

(5) Datum systems other than NAD 27/NAD 83 will be used in OCONUS locations. Selected military operational requirements in CONUS may also require non-NAD datum references. It is recommended that GPS baselines be directly adjusted on the specific project datum. Most survey-grade GPS receivers and adjustment software is capable of working in any worldwide ellipsoid or datum.

d. Connections to existing control. A variety of methods are available for making accurate connections and adjustments to the NGRS, such as OPUS, Auto-GIPSY, and SCOUT, which are described in Chapter 10. For most static and kinematic GPS horizontal control work, at least two (and preferably three) existing control points should be connected for referencing and adjusting a new GPS survey (Table 8-2). For higher-order NGRS densification, Table 2 of FGCC 1988 contains additional network connection criteria. Programs such as OPUS will adjust observations to three nearby NGRS/CORS stations. Existing points may be part of the NGRS (e.g., CORS stations) or in-place project control that has been adequately used for years. Additional points may be connected if practical. In some instances, a single existing point may be used to generate spurred baseline vectors for supplemental construction control. For higher-order work, Table 8-2 recommends a baseline check connection between existing control points. This would not apply to NGRS CORS stations.

(1) Connections with local project control. The first choice for referencing new GPS surveys is the existing project control. This is true for most surveying, not just GPS, and has considerable legal basis. Unless a newly authorized project is involved, long-established project control reference points should be used, or at a minimum, connected with the overall scheme. If the project is currently on a local datum, then a supplemental tie to the NGRS should be considered as part of the project.

(2) Connections with NGRS. Connections with the NGRS are preferred where prudent and practical. As with conventional USACE surveying, such connections to the NGRS are not mandatory. In many instances, accurate connections with the NGRS CORS points may be easily (and economically) performed using NGS OPUS software methods--see Chapter 10. When existing project control is known to be of poor accuracy, then ties (and total readjustment) to the NGRS may be warranted. Sufficient project funds should have been programmed to cover the additional costs of these connections, including data submittal and review efforts if such work is intended to be included in the NGRS.

(3) Mixed NGRS and project control connections. On existing projects, NGRS-referenced points should not be arbitrarily mixed with existing project control. This is especially important if existing project control was poorly connected with the older NAD 27 control, or if the method of this original connection is uncertain. Since NGRS control has been readjusted to NAD 83 (including subsequent high-precision HARNS readjustments of NAD 83), and most USACE project control has not, problems may result if these schemes are mixed indiscriminately. If a decision is made to establish and/or update control on an existing project, and connections with the NGRS (NAD 83 (xxxx)) are required, then all existing project control points must be resurveyed and readjusted. Mixing different reference systems can result in different reference datums, with obvious adverse impacts on subsequent construction or boundary reference. It is far preferable to use "weak" existing (long-established) project control (on NAD 27 or whatever datum) for reference than to end up with a mixture of different systems or datums. See EM 1110-1-1004 for further discussion.

(4) Accuracy of connected reference control. Ideally, connections should be made to control of a higher order of accuracy than that intended for the project. In cases where NGRS/CORS control is readily available, this is usually the case. However, when only existing project control is available, connection and adjustment will have to be performed using that reference system, regardless of its accuracy. GPS baseline measurements should be performed over existing control to assess its accuracy and adequacy for adjustment, or to configure partially constrained adjustments.

(5) Connection constraints. Although Table 8-2 requires only a minimum of two existing stations to reliably connect GPS static and kinematic surveys, it may often be prudent to include additional NGRS and/or project points, especially if the existing network is of poor reliability. Adding more NGRS points provides redundant checks on the surrounding network. This allows for the elimination of these points should the final constrained adjustment indicate a problem with one or more of the fixed points. Use of NGS OPUS adjustment techniques allows DGPS connection and adjustment with multiple high-accuracy CORS stations, with positional accuracies within the 2 cm level. Table 8-2 also indicates the maximum allowable distance GPS baselines should extend from the existing network. FGCC 1988 standards also require connections to be spread over different quadrants relative to the survey project. This is recommended if possible; however, such requirements are usually unnecessary for most USACE work.

e. Multiple/Repeat/Independent baseline connections. FGCC 1988 lists recommended criteria for baseline connections between stations, repeat baseline observations, and multiple station occupations. Many of these standards were developed by NGS for performing high-precision geodetic control surveys such that extensive redundancy will result from the collected data. Since the purpose of these geodetic densification surveys is markedly different from USACE control densification, the need for such high observational redundancy is also different. Therefore, Table 8-2 recommends not less than two repeat baseline occupations in a project--or at least 10% of the baselines in a project be double observed--whichever is less. Adding redundant baseline occupations may prove prudent on some remote projects where accessibility is difficult. In addition, observing the same baselines at different times and satellite configurations provides a good quality control check. When vertical control densification is performed, then all baselines shall be observed at least twice.

f. Independent baselines. When 3 or more receivers simultaneously occupy stations on a network, baselines can be formed from each pair of receivers. However, not all of these baselines are independent or "nontrivial." The "dependent" baselines are considered as "trivial" since they do not provide a unique solution. For example, if 4 receivers are deployed, 6 baselines are formed by these receivers. Only 3 of these lines are "independent"--the other 3 are "trivial" or dependent. The number of total baselines and independent baselines can be computed from the following:

$$\text{Number of Baselines per Session} = N (N - 1) / 2$$

$$\text{Number of Independent Baselines per Session} = N - 1$$

where N = number of receivers deployed on network. If only 2 receivers are used, then all the baselines will be independent. If 4 receivers are used over 10 sessions, then 30 independent baseline sessions would result. During the baseline processing and adjustment phase, the surveyor must identify the independent baselines when trivial observations are present, and ensure the adjustment statistics do not double count (or erroneously over weight) trivial baselines.

g. Loop requirements. Loops (i.e. traverses) made from GPS baseline observations provide the mechanism for performing field data validation as well as a final adjustment accuracy analysis. Since loops of GPS baselines are comparable to terrestrial EDM traverse routes, misclosures and adjustments can be handled similarly. Most GPS survey nets (static or kinematic) end up with one or more interconnecting loops that are either internal from a single fixed point or external through two or more fixed network points. Loops should be closed off within the maximum number of station intervals indicated in Table 8-2. Loop closures should meet the criteria specified in Table 8-2, based on the total loop length that should also be kept within the maximum lengths shown in Table 8-2. Loops must not include trivial baselines.

(1) GPS control surveys should be conducted by forming loops between two or more existing network control points, with adequate cross-connections within the loops where feasible. Such alignment procedures are usually most practical on civil works navigation projects, which typically require control to be established along a linear path, e.g., river or canal embankments, levees, beach renourishment projects, and jetties. Connections to existing control should be made as opportunities exist and/or as often as practical.

(2) When establishing control over relatively large military installations, civil recreation projects, flood control projects, and the like, a series of redundant baselines forming interconnecting loops is usually recommended. When densifying Second- and Third-Order control for site plan design and construction, extensive cross-connecting loop and network configurations recommended by the FGCC 1988 for geodetic surveying are not necessary.

(3) On all projects, maximum use of combined static and kinematic GPS observations should be considered, both of which may be configured to form pseudo-traverse loops for subsequent field data validation and final adjustment.

8-8. Recommended Static Baseline Occupation Times for Horizontal Control Surveys

Station occupation time is dependent on baseline length, number of satellites observed, GDOP, and the GPS equipment used. Occupation must be long enough to reliably fix the integer ambiguity in the baseline solution; thus, the more satellites in view the more reliable and faster the integers can be fixed. In general, a 20-minute to 2-hour occupation is required for lines less than 50 km. A rough guideline for estimating static baseline occupation time is shown in Table 8-3 below. From a statistical perspective, lengthier occupation times may not necessarily improve the accuracy once the integers have been reliably fixed. Reobserving the baseline on a different day/time (i.e. over a different satellite configuration) will provide better redundancy.

Table 8-3. Guidelines for Determining Static Baseline Occupation Time versus Satellite Visibility and Baseline Length--Single- and Dual-Frequency Horizontal GPS Control Surveys

	Recommended Minimum Observation Time (minutes) Satellites in View/Single- or Dual-Frequency Receiver					
	4		5		6 or more satellites in view	
Baseline Length (km)	Single	Dual	Single	Dual	Single	Dual
1-10 km	60 min	20 min	36 min	12 min	24 min	8 min
10-20 km	75 min	25 min	45 min	15 min	30 min	10 min
20-50 km [1]	105 min	35 min	75 min	25 min	60 min	20 min
> 50 [1]	180 min	60 min	135 min	45 min	90 min	30 min

[1] Dual-frequency receivers are recommended for baselines greater than 20 km
Source: USACE GPS Field Review Group (September 2002)

Some software vendors recommend shorter observation times when conditions are ideal (i.e. clear, unobstructed horizons, good quality measurements, dual-frequency geodetic quality receivers, and good geometry). For example, Waypoint Consulting recommends the following "rule-of-thumb" for determining the observation time on a baseline:

Baseline Observation Time = 10 minutes + 1 minute/km (Single frequency)

Baseline Observation Time = 5 minutes + 0.5 minute/km (Dual frequency)

The above guideline presumes expected horizontal accuracies of 10 mm, clear visibility, and clean data. Thus for a 40 km line, a dual-frequency minimum observation time would be 25 minutes. Alternatively, when precise vertical control is being densified using GPS, then session lengths may need to be increased--including observation of redundant baselines on different days.

a. Caveats. Due to the multitude of variables inherent in GPS surveying, there is no exact formula for determining the required baseline occupation time. The values shown in Table 8-3 are only general guidelines. The results from the baseline reduction (and subsequent adjustments) will govern the adequacy of the observation irrespective of the actual observation time. The most prudent policy is to exceed the minimum recommended times, especially for lines where reoccupation would be difficult or field data assessment capabilities are limited. Local conditions, manufacturer recommendations, and personal experience with specific receiver capabilities and baseline reduction results should also be factored into baseline occupation time requirements.

b. Dual-frequency receivers. For baselines greater than 10 km in length, the ionosphere usually has an adverse effect on the solution. Adverse ionosphere effects for baselines of this length can be reduced by using a dual-frequency GPS receiver, resulting in a more accurate ionospheric-free fixed solution than the less accurate float solution that might result from a single-frequency receiver. Accordingly, Table 8-3 recommends that dual-frequency receivers be used for baselines over 20 km in length. This is because fixed solution integers become more difficult to solve as the baselines lengthen. The dual-frequency receiver also provides "wide laning," which is a combination of the L1 and L2 frequencies. Wide laning is used to search and resolve the integer ambiguities.

8-9. Network Design and Layout for Carrier Phase GPS Horizontal Control Surveys

A wide variety of survey configuration methods may be used to densify project control using static and/or kinematic GPS survey techniques. Unlike terrestrial triangulation, trilateration, and EDM traverse surveying, the shape, or geometry, of the GPS network design is usually not as significant. The following guidelines for planning and designing proposed GPS surveys are intended to support lower order (Second-Order, Class I, or 1:50,000 or less accuracy) control surveys applicable to USACE civil works and military construction activities. An exception to this would be GPS surveys supporting structural deformation monitoring projects where relative accuracies at the centimeter level or better are required over a small project area.

a. NGRS connections. Newly established GPS control may or may not be incorporated into the NGRS, depending on the adequacy of connection to the existing NGRS network, or whether it was tied only internally to existing project control.

b. Project accuracy requirements. Of paramount importance in developing a network design is to obtain the most economical coverage within the prescribed project accuracy requirements. The optimum network design, therefore, provides a minimum amount of baseline/loop redundancy without an unnecessary amount of "over-observation." Obtaining this optimum design (cost versus accuracy) is difficult and constantly changing due to evolving GPS technology and satellite coverage.

c. GPS survey network schemes. Planning a GPS survey network scheme is similar to that for conventional triangulation or traversing. The type of survey design adopted is dependent on the GPS technique employed and the requirements of the user. A GPS network is developed to extend project control over an area. The network design establishes the stations to be occupied (new and existing) and specific baselines to be observed. The network design also includes the GPS observing sequence with a given number of GPS receivers. In addition, the network design should be geometrically sound and meet the criteria in Table 8-2. Triangles that are weak geometrically should be avoided, if possible. For lower-order work, elaborate network design schemes are unnecessary and less work-intensive GPS survey extension methods may be used. Care must be taken to avoid including trivial baselines in the final network adjustment. For high-accuracy vertical densification projects, duplicate or redundant baseline occupations may be required. The following figures depict examples of step-by-step methods to build a

GPS survey network, with a given number of receivers. Other combinations of observing sessions could be developed to accomplish the same results. The network consists of three fixed (known X-Y-Z) control points shown by circled triangles. The three unknown points are shown by triangles. Solid connecting lines are observed baselines in a session. Baselines marked by "t" are trivial baselines that should be excluded in any network adjustment.

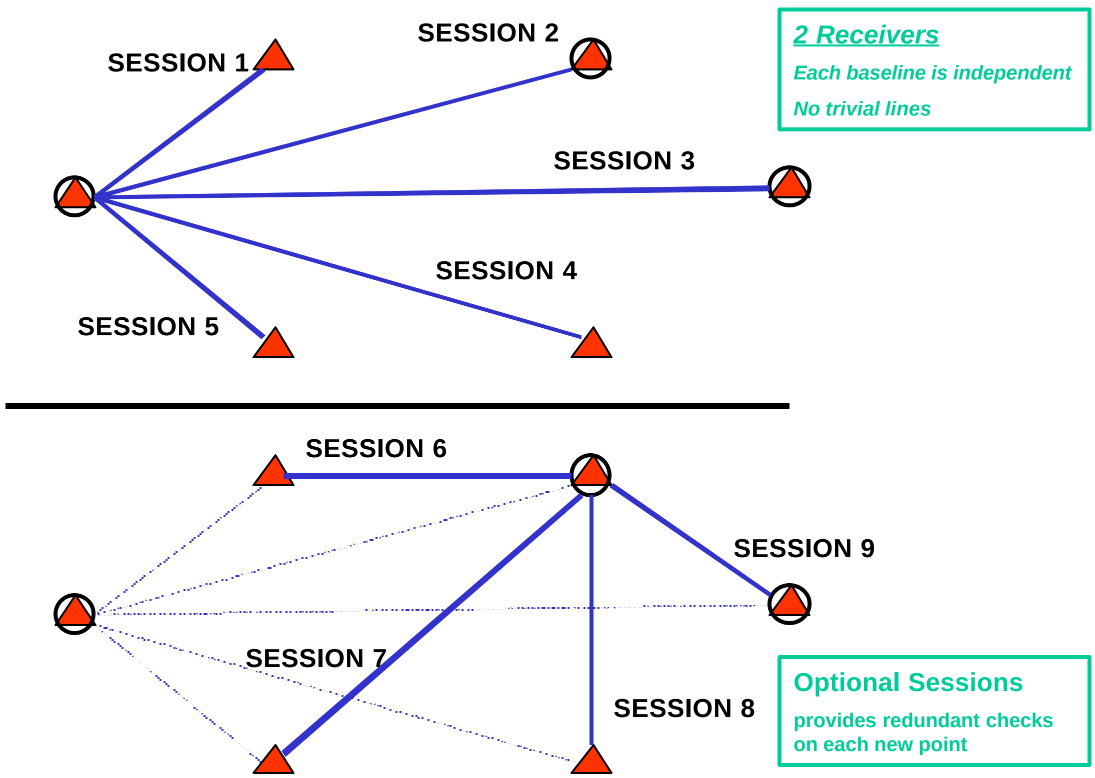


Figure 8-6. GPS observing sessions for 2 GPS receivers. If a check on the spurred positions is required, then sessions 6 through 9 could be optionally added.

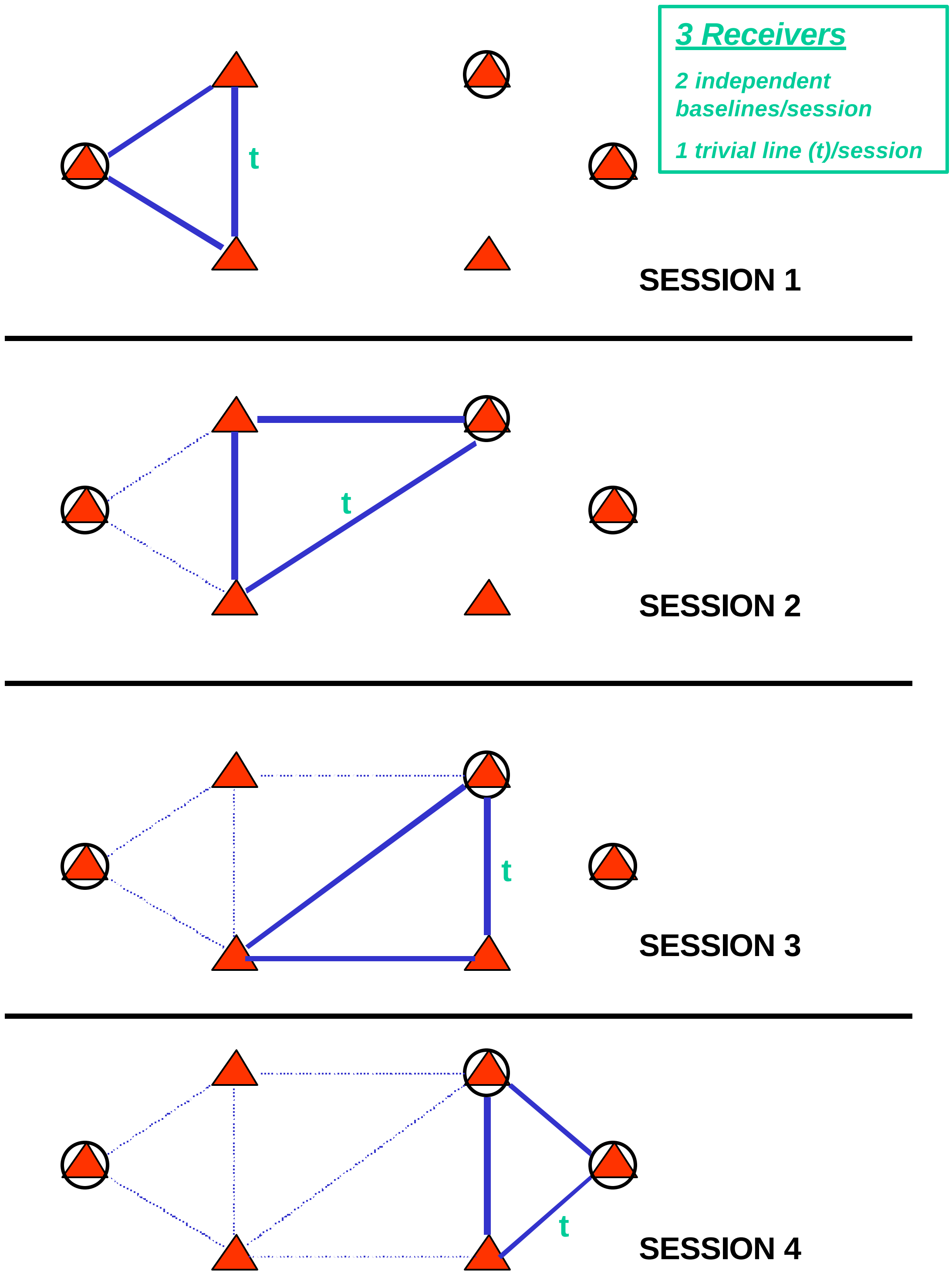


Figure 8-7. GPS network and observing session design given 3 GPS receivers

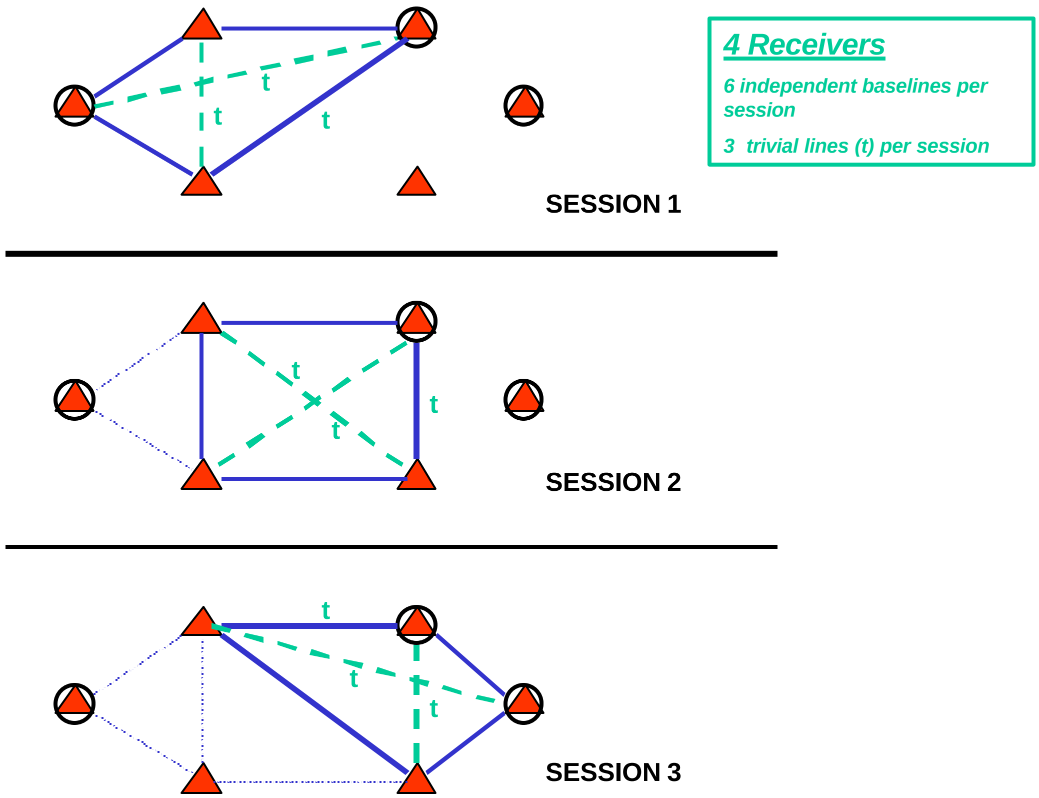


Figure 8-8. GPS network and observing session design given 4 GPS receivers

d. GPS Traversing. Traversing is the method of choice when the user has only two or three receivers and required accuracies are 1:5,000-1:50,000. Traversing with GPS is done similar to conventional methods. Open-end traverses are not recommended when 1:5,000 accuracies or greater are required. A minimum of one fixed (or known) control point is required, although two or more are preferred for redundancy. These points may or may not be part of the NGRS, or they may be existing Corps project control monuments. A closed loop traverse between two points is always preferred, as shown in Figure 8-9. When performing a loop traverse to/from a single point (open traverse or loop traverse), the surveyor should observe a check angle or check azimuth at the known point using conventional survey techniques to determine if the station has been disturbed.

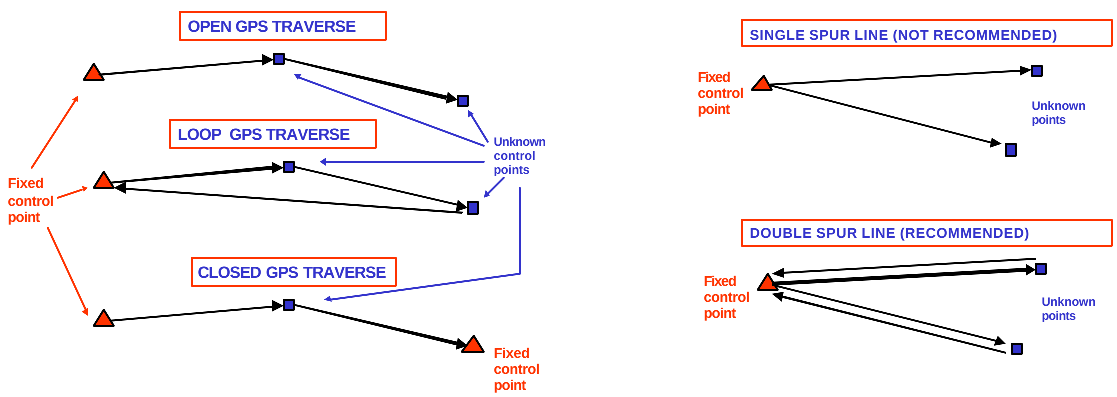


Figure 8-9. GPS traversing and spur line observation schemes

e. GPS spur shots. Spurs (or radial observations) are an acceptable method when the user has only two receivers or only a few lower-order project control points are to be established. Spur baselines should be observed twice during two independent observing sessions. Once the first session is completed, the receivers at each station must be turned off and the tripods moved and replumbed. Preferably, the second session should be observed during a different phase of the satellite constellation. This procedure is similar to performing a forward and backward level line. If this step is not implemented, the two baselines cannot be considered independent. Figure 8-9 above shows an example of a double observation over a spur line. The single spur shots shown in Figure 8-9 are not recommended for primary control; however, they are acceptable for standard site plan topographic and feature mapping typically performed with total station or RTK GPS methods.

8-10. Planning Vertical Control Surveys using Carrier Phase GPS

GPS can be used to extend and densify precise vertical control on USACE civil and military projects. This GPS application requires far more exacting measurement procedures than that required for horizontal control densification. This is due to: (1) GPS is less accurate in the vertical axis, (2) local geoid undulations must be accounted for, (3) the variety of vertical datum definitions and local readjustments, and (4) vertical orthometric datums can exhibit significant short-term local variations due to settlement. Accordingly, planning considerations for vertical control surveys are more critical than those required for horizontal surveys.

a. GPS positioning, whether operated in an absolute or differential positioning mode, can provide heights (or height differences) of surveyed points. As outlined in Chapter 3, the height " *h* " or height difference " *Dh* " obtained from GPS is in terms of height above or below the WGS 84 ellipsoid. These ellipsoid heights are not the same as orthometric heights, or elevations, which would be obtained from conventional differential/spirit leveling. This distinction between ellipsoid heights and orthometric elevations is critical to USACE engineering and construction projects; thus, users of GPS must exercise extreme care and caution in applying GPS height determinations to projects that are based on conventional orthometric elevations.

b. GPS uses WGS 84 as the optimal mathematical model best describing the shape of the true earth at sea level based on an ellipsoid of revolution. The WGS 84 ellipsoid adheres very well to the shape of the earth in terms of horizontal coordinates, but differs somewhat with the established definition(s) of orthometric height. The difference between ellipsoidal height, as measured by GPS, and conventional leveled (orthometric) heights is required over an entire project area to adjust GPS heights to orthometric elevations. In planning a vertical control project, appropriate geoid modeling software must be used to convert ellipsoidal heights to approximate orthometric elevations. These approximate geoid model conversions have improved with each release by NGS but should also be used with care and caution. In practice, conventional spirit leveling is performed as a quality control process for these geoidal adjustments.

c. Static or kinematic GPS survey techniques can be used effectively on a regional basis for the densification of lower accuracy vertical control for mapping purposes. Existing benchmark data (orthometric heights) and corresponding GPS-derived ellipsoidal values for at least three stations in a small project area can be used in tandem in a minimally constrained adjustment program to reasonably model the geoid in the local project area. More than three correlated stations are required for larger areas to ensure proper modeling of the geoidal undulations in the area. The model from the benchmark data and corresponding GPS data can then be used to derive the unknown orthometric heights of the remaining stations occupied during the GPS observation period.

d. Step-by-step vertical control planning, observation, and adjustment procedures employed by the NGS are described in the following sections and in some of the publications listed in Appendix A. These procedures are recommended should a USACE field activity utilize GPS to densify vertical control relative to an orthometric datum.

e. The criteria in the following sections do not apply to RTK topographic mapping surveys performed over a relatively small distance (say 500 to 1,000 m) from an existing benchmark. Over these short distances, geoid undulations are usually insignificant and observed ellipsoidal elevation differences can be assumed as orthometric. This is not the case when RTK surveys are extended beyond these distances, such as those typically encountered in determining elevations of dredge dragarms, core drilling rigs, or hydrographic survey platforms in distant, offshore navigation projects. For these applications, geoid modeling must be determined and applied.

8-11. Technical Criteria for GPS Vertical Densification

The following sections provide technical guidance for using differential carrier-phase GPS observations to determine elevations of survey benchmarks for wide-area mapping and GIS database development applications. Recommended procedural specifications for field DGPS observation sessions are included. These guidelines and specifications are intended for densifying vertical control over large project areas, such as an entire military installation or watershed basin mapping project. The DGPS methods outlined in this section are generally not intended, nor would be cost-effective, for small projects or any type of construction lay out work where vertical grades or benchmarks require an accuracy better than 30 millimeters (mm). In such cases, conventional differential (spirit) leveling methods should be used. Advances in geoid modeling have also led to more accurate conversions between NAD 83 GPS ellipsoid heights and NAVD 88 orthometric height systems. Accuracies of 30 mm or better have been obtained when converting ellipsoid heights from GPS surveys, based on NAD 83 control, to NAVD 88 orthometric heights using the latest geoid model. The initial GPS survey data must be valid for the elevation transfer method to be effective.

8-12. Orthometric Elevation Guidelines

The following paragraphs present the basic criteria standards for using GPS to determine NAVD 88 elevations. These criteria are also summarized in Table 8-4. These operational requirements are based on field test results conducted by US Army Topographic Engineering Center (ERDC/TEC) and the National Geodetic Survey (NGS) using several different GPS surveying methods and comparing these results to conventional differential leveling networks. Additional standards and specifications for performing precise GPS vertical control surveys are contained in *Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm)* (NOAA 1997). Much of the guidance contained in this section is derived from the material in this reference. USACE commands performing vertical densification surveys with GPS should be thoroughly familiar with the contents of the NOAA 1997 reference.

Table 8-4. Guidelines for Establishing GPS-Derived ± 30 mm Accuracy Orthometric Elevations

Occupation time for each baseline occupation (minimum):

Distance	Time	Update rate
< 10 km	30 min	5 sec intervals
10-20 km	60 min	10 sec intervals
20-40 km	120 min	15 sec intervals
40-60 km	180 min	15 sec intervals
60-80 km	240 min	15 sec intervals
80-100 km	300 min	15 sec intervals
> 100 km	> 5 hours	15 sec intervals

Dual-frequency receiver required:	Yes
Geodetic quality antenna with ground plane required:	Yes
Minimum number of existing benchmarks required:	3
Minimum number of observations per baseline:	2
Fixed-height tripods/poles:	Recommended
Measure antenna height:	2 to 3 times
Satellite altitude mask angle:	15 degrees
Maximum allowable VDOP:	5
Number of days station occupied:	2 days
Over 40 km baselines:	3 days
Nominal distance between project and fixed, higher-order benchmarks:	within 20 km radius
Maximum distance between same or higher-order benchmarks:	50 km
Collect meteorological data:	Optional
Precise ephemeris baseline reduction required:	Yes
Recommended geoid model:	Geoid 99 (or most recent)
Fixed integers required for all baselines:	Yes
Baseline resultant RMS less than:	2.5

Source: Table 1 of (NOAA 1997) with USACE revisions *a. Baselines.* GPS baselines are observed to determine ellipsoidal height differences across a network of stations. A GPS precise ephemeris should be used in place of the broadcast ephemeris during baseline data processing. Repeat baselines should be observed for all control surveys established with DGPS. The average ellipsoid height from the repeat observations will be closer to the truth, with a few exceptions, than the ellipsoid height value from a single observation. Table 8-4 recommends a minimum of two repeat observations for each baseline. Baselines should be reobserved on different days with significantly different satellite geometry. For topographic and location surveys (using kinematic techniques), repeat occupations should be performed where feasible. It is important that the positions be adjusted on NAD 83 since the most recent geoid models are also based on NAD 83.

b. NAVD 88 network connections. At least three or more established NAVD 88 First-Order benchmarks should be occupied to serve as the GPS reference stations where accurate vertical coordinates will be fixed for the network adjustment. It is suggested that at least one (preferably 2 or more) of these benchmarks are also High Precision Geodetic Network (HPGN), High Accuracy Reference Network (HARN), or CORS stations to ensure accurate geoid modeling. First-Order accuracy standards for geodetic leveling ensure the relative vertical position of these reference monuments will agree. Redundant vertical control within the project area will provide a check on the solution heights of the unknown stations. The ideal condition would be to have all benchmarks with high-order vertical and horizontal control surrounding and/or within the project area. Table 8-4 recommends the distance between existing reference benchmarks should be kept within 50 km. If this value is exceeded, then additional network connections should be added.

c. Geoid models. Geoid heights at the reference stations are determined from a published geoid model, GEOID 96 or GEOID 99. The geoid height is added to the published orthometric height at the GPS reference station to determine its ellipsoid height to the accuracy level of the geoid model. Once the reference stations' ellipsoidal, orthometric and geoidal heights have been fully determined, elevations are transferred from the reference stations to the remaining points in the network according to the following relations:

From Equation 3-1 back in Chapter 3,

$H_i = h_i - N_I$ (where i is the station of unknown height)

$H_{ref} = h_{ref} - N_{ref}$, (where ref is the station of known heights)

with measured difference in ellipsoid height ($Dh = h_i - h_{ref}$) from a DGPS survey, and computed difference in geoid height ($DN = N_i - N_{ref}$) from a known geoid model, then,

$$H_i = H_{ref} + (H_i - H_{ref})$$

$$H_i = H_{ref} + (h_i - N_i) - (h_{ref} - N_{ref})$$

$$H_i = H_{ref} + (h_i - h_{ref}) - (N_i - N_{ref})$$

Then,

$$H_i = H_{ref} + (Dh - DN) \qquad \text{(Eq 8-1)}$$

where in Equation 8-1 H_i is the orthometric height of the *i-th* station, the quantity Dh is determined from the measured GPS ellipsoidal height differences, and the quantity DN is the geoidal height difference computed from the geoid model. Over very small distances (< 1,000 m), DN may be considered negligible, and the ellipsoidal height difference Dh is added directly to the orthometric height of the reference station to obtain the orthometric height of the unknown point. This practice is commonly employed for GPS RTK topographic site plan or construction surveys over small areas.

d. Elevation precision and accuracy. The expected precision of the orthometric height from using GPS relative positioning, modeled geoid heights, and the above relation in Equation 8-1 can be calculated by the summation of variance components corresponding to the accuracy of the published orthometric height, the GPS relative height determination, and the computed geoid height differences. Positional accuracy for orthometric heights on benchmarks must be obtained from published sources based on the results of a vertical network adjustment. Without this information it is presumed that a fixed vertical control point contributes no additional error to the height of the unknown stations. The uncertainties in GPS relative heights are estimated from the vertical component error estimate that is produced from the GPS data processing and adjustment software. An error estimate of ±10 mm is commonly seen as the minimum baseline error produced from static type surveys. Relative geoidal height precision ($\sigma_{\Delta N}$) from geoid modeling can have an expected standard deviation of between ±10 mm and ±20 mm.

e. Elevation confidence. The aforementioned error values lead to an expected uncertainty in final orthometric height at the unknown station of approximately 3 cm (at the 95% confidence level) relative to the published elevation at the benchmark reference station. NOAA 1997 specifies procedures for two potential levels of accuracy: ±20 mm and ±50 mm. NOAA procedures for "±2 cm (20 mm)" ellipsoid elevation accuracy should be followed if approximately ±30 mm accuracy reduced orthometric elevations are desired. A repeatable accuracy of ±30 mm meets or exceeds most feature elevation tolerances specified for many USACE surveying and mapping projects, excepting certain high precision surveys such as for low flow hydraulic studies, construction stake out, or structural deformation monitoring networks. In areas with obstructions, dense vegetation, or high relief between monuments or projects site, GPS may be the most cost effective approach and may exceed spirit leveling accuracy. In some cases, GPS elevation difference observations can be obtained more quickly than conventional differential leveling observations.

f. Field testing results. Based on an evaluation of DGPS data and geoid modeling software capabilities by ERDC/TEC, it was determined that higher accuracy elevations are obtained by the transfer of ellipsoidal height differences and relative geoidal heights from a station with a known NAVD 88 elevation, than is possible from the direct application of absolute geoid heights to GPS networks. This analysis was based on various methods used for determining NAVD 88 elevations from GPS ellipsoidal height data. These methods were tested on a network of points having known First-Order leveled orthometric heights that were tied to First-Order vertical control. Results of the testing indicated that GPS-based surveys could determine NAVD 88 elevations to an accuracy of ±30 mm when relative heights and differences in geoid heights are applied. It is important to note that the accuracy of NAVD 88 elevations determined from DGPS-derived heights and geoid modeling is dependent on the accuracy of the GPS coordinate solution and the geoid model.

8-13. Additional Guidelines and Recommendations for Planning GPS Vertical Densification

In addition to the guidelines presented in the above section, the following procedures and methods are recommended and should be implemented when planning to use GPS for elevation determination.

a. Keep project areas within a 20-kilometer radius of control points. GPS relative positioning accuracy depends in part on the length of the measured baseline. Positioning errors grow in direct proportion to baseline length at a rate of approximately 1 part per million. For networks with an area less than 20 km, the distance dependent error in the GPS vertical component (relative ellipsoid height) will be limited. Occupation times of less than 1 hour (see Table 8-4) should produce good results for these shorter baselines. For project areas greater than 20 km, the occupation times should be increased to a minimum of 2 hours. Fixed control points should be spaced throughout (surrounding and within) the project area.

b. Observe when VDOP is less than 5.0. Vertical Dilution of Precision (VDOP) is a measure of vertical positioning accuracy (due mainly to satellite geometry) relative to the precision of the measurements used to determine the position. Large VDOP values represent poor satellite geometry that will generally produce weak positioning solutions.

c. Use fixed-height tripods/poles. Fixed-height tripods and range poles provide a consistent station occupation method that can reduce the likelihood of antenna height measurement blunders.

d. Use dual-frequency receivers. Dual-frequency receivers can correct GPS measurements for ionospheric-based range errors. This will extend the feasible baseline length and resolve integer ambiguities reliably within 20 km. Dual-frequency receivers should be used on all baselines longer than 20 km.

e. Use identical geodetic quality antennas with ground plane. Different makes and models of GPS antennas can have different phase centers. Mixing of different types of antennas can cause errors in the vertical component up to 100 mm. Only if the processing software can account for the phase center difference in the GPS antennas should mixing of antenna types occur. The ground plane on the antenna (or choke ring antenna design) will reduce the amount of ground reflecting multipath.

f. Occupy points a minimum of twice with different satellite constellations and on different days. The purpose of this criteria is to ensure different atmospheric conditions (different days) and significantly different satellite geometry (different times) for the two occupations. For example, if the first day's observation was made between 8:00 AM to 8:30 AM, the second 30-minute observation would be made on the next day anytime between 11:30 AM and 5:30 PM. If the second observation is not made for a couple of days or even a week, be sure to compensate for the daily 4-minute accumulative change in the GPS satellite constellation. It has been shown that the average ellipsoid height of repeat observations is closer to the truth, with a few exceptions, than the ellipsoid height of a single observation.

g. Process with a minimum elevation mask of 15 degrees. A 15-degree elevation mask will reduce noise embedded in low elevation satellite data and also minimize potential multipath effects from nearby objects surrounding the antenna. For obstructions low on the horizon, a 20-degree elevation mask may be used during baseline processing.

h. Process GPS data with Precise Ephemeris. The broadcast ephemeris is the prediction of where the satellites will be, but the precise ephemeris is the actual true orbit of the satellites. Use of a precise ephemeris will reduce the error between predicted and actual satellite orbit and increase the accuracy of the survey. The precise ephemeris is available approximately seven days after a survey through the National Geodetic Survey.

i. Use only ionosphere free fixed baseline solutions for baselines greater than 10 k. Ionosphere-free solutions indicate the use of dual-frequency receivers and processing can model and eliminate errors due to signal delay in the ionosphere. Fixed baseline solutions indicate a statistically accurate integer ambiguity was established from the GPS data. A normal, (not ionosphere-free) fixed baseline is sufficient for baselines less than 10 km.

j. Use relative geoid height values. Application of the geoid model to both reference and remote stations will produce two absolute geoid heights. The relative geoid height value is determined from the difference between the absolute geoid model height values taken at both ends of a given baseline. Relative geoid heights, when added to measured ellipsoidal height differences, produces the best vertical accuracy based on the ground truth test results.

k. Adjustments. A minimally constrained least-squares adjustment should be performed on the vertical reference network to determine which of the "fixed" benchmarks are valid. Such a free adjustment would hold one of the benchmarks fixed in X, Y, and Z in order to check the fit against the other established benchmarks. Presuming no observational blunders, any benchmark with apparent excessive movement would be discarded from the final (constrained) adjustment.

l. Geoid models. Different geoid models should not be mixed in the same project. Different geoid models can vary by 5 cm or more. For example, if a project network was originally adjusted using GEOID 93, do not mix in GEOID 96 or GEOID 99 with subsequent observations--unless the entire project is readjusted using a later geoid model. GEOID 96 (or a more recent update) is recommended for projects on NAVD 88 datum.

8-14. Cadastral Survey Standards and Guidelines using GPS

The Bureau of Land Management and the US Forest Service jointly developed GPS survey standards for surveys of the public lands of the US-- *Standards and Guidelines for Cadastral Surveys using Global Positioning System Methods* (USFS/BLM 2001). These standards and guidelines may prove useful when Corps commands are required to connect military installation boundaries or reservoir boundaries with the US Public Land Survey System (PLSS). Both static and kinematic survey techniques are covered in the guidelines. They also cover field data acquisition methods, field survey operation and procedures, data processing and analysis methodologies, and required documentation. Two types of GPS control surveys are defined: (1) Cadastral Project Control, and (2) Cadastral Measurements.

a. Cadastral Project Control includes monuments established by direct connection with the primary NGRS (HARN/HPGN/CORS) network. These monuments serve as the basis for all subsequent connections by GPS Cadastral Measurements made to PLSS monuments. Cadastral Project Control must be connected with at least two NGRS points. The reference datum shall be the latest epoch of NAD 1983 (1986)--e.g., Wyoming NAD 1983 (1993). Only Static or Fast-Static survey methods are allowed for these connections. Points must be established by two or more independent baselines, loops must have a minimum of three baselines, baseline solutions must be fixed double difference, and all stations must have at least two independent occupations. Single radial (spur) baselines are not allowed.

b. Cadastral Measurements are used to define the location of PLSS corners and boundaries. Cadastral Measurements must be connected with at least two Cadastral Project Control monuments or NGRS monuments. Guidelines for these observations are similar to those required for Cadastral Project Control, except all types of static and kinematic survey methods are acceptable, including real-time kinematic RTK) techniques. The USFS/BLM guidelines contain extensive procedural and calibration requirements for RTK surveys.

c. Positional accuracy standards for USFS/BLM cadastral surveys are defined relative to the 95% confidence level, as outlined in Table 8-5 below.

Table 8-5. USFS/BLM Cadastral Survey Standards for Positional Accuracy

	Definition	95 % Confidence Level Cadastral Project Control	Cadastral Measurements
Local Accuracy	Average measure of the relative accuracies of the coordinates for a point with respect to other adjacent points	0.050 m	0.100 m
Network Accuracy	Relative to the NGRS network	0.100 m	0.200 m

The 95% positional accuracy of established points is assessed from the output of the network adjustment, as explained in Chapter 11.

8-15. Field Planning Considerations for GPS Surveys

After a GPS horizontal and/or vertical densification network has been designed, specified, and laid out, the logistics of performing the GPS field survey needs to be considered. The most efficient survey method should be chosen in order to minimize time and cost while meeting the accuracy requirements of a given survey project. Once a survey technique is developed, equipment requirements, personnel assignments, observation schedules, and session designations can be identified.

a. General equipment requirements. The type of GPS instrumentation required for a survey depends on the accuracy requirements of the project, GPS survey technique, project size, and economics. Most USACE projects can be completed using a single-frequency receiver. Dual-frequency receivers are recommended as baseline lengths approach or exceed 20 km. This length may also vary depending on the amount of solar activity during the observation period. Using a dual-frequency receiver permits the user to solve for possible ionospheric and troposphere delays, which can occur as the signal travels from the satellite to the receiver antenna.

(1) Number of GPS receivers. The minimum number of receivers required to perform a differential GPS survey is two. The actual number used on a project will depend on the project size and number of available instruments/operators. Using more than two receivers will often increase productivity and allow for more efficient field observations. For some post-processed kinematic applications, two reference receivers (set at known points) and at least one rover are recommended.

(2) Personnel. Personnel requirements are also project dependent. Most GPS equipment is compact and lightweight and only requires one person per station set-up. However, in some cases where a station is not easily accessible or requires additional power for a data link, two individuals may be required.

(3) Transportation. One vehicle is normally required for each GPS receiver used on a project. If secure sites are available, GPS receivers may be left unattended. The survey vehicle should be equipped to handle the physical conditions that may be encountered while performing the field observations. In most cases, a two-wheel-drive vehicle should be adequate for performing all field observations. If adverse site conditions exist, a four-wheel-drive vehicle may be required. Adequate and reliable transportation is important when the observation schedule requires moving from one station to another between observation sessions.

(4) Auxiliary equipment. Adequate power should be available for all equipment (receivers, computers, lights, etc.) that will be used during the observations. Computers, software, and data storage/archiving devices should be available for on-site field data reduction use. Other survey equipment should include tripods, tribrachs, tribrach adapters, radios, cell phones, measuring tapes, flagging, flashlights, tools, equipment cables, compass, psychrometer, inclinometer, etc. If real-time positioning is required, than a data link is also needed.

(5) Benchmarks. Special equipment is required to set deep-driven permanent benchmarks, as illustrated in Figure 8-10.

Figure 8-10. Setting deep-driven benchmarks (Memphis District & 3001, Inc.)

b. Observation schedules. Planning a GPS survey requires that the surveyor determine when satellites will be visible for the given survey area; therefore, the first step in determining observation schedules is to plot a satellite visibility plot for days GPS observations are planned. If some sectors are obstructed, at least 4 satellites may not be visible at all times. At least 5 satellites are required for RTK OTF initialization.

(1) Most GPS manufacturers have software packages that predict satellite rise and set times. Satellite predictions are also available on various web sites. A satellite plot will have the following essential information: satellite azimuths, elevations, set and rise times, and PDOP for the desired survey area. A typical visibility plot is shown at Figure 8-11. Satellite ephemeris data are generally required as input for the prediction software.

(2) To obtain broadcast ephemeris information, a GPS receiver collects data during a satellite window. The receiver antenna does not have to be located over a known point when collecting a broadcast ephemeris. The data is then downloaded to a personal computer where it is used as input into the software prediction program. Besides ephemeris data for the software, the user is generally required to enter approximate latitude and longitude (usually scaled from a topographic map) and time offset from UTC for the survey area. A current ephemeris file can be downloaded using various manufacturer's planning software.

(3) From the satellite plot, the user can determine the best time to perform a successful GPS survey by taking advantage of the best combination of satellite azimuths, elevations, and PDOP as determined by the satellite visibility plot for the desired survey area. The number of sessions and/or stations per day depends on satellite visibility, travel times between stations, and the final accuracy of the survey. Often, a receiver is required to occupy a station for more than one session per day.

(4) A satellite visibility plot and a PDOP versus time plot may be run prior to site reconnaissance. The output files created by the satellite prediction software are used in determining if a site is suitable for GPS surveying.

(5) Determination of session times is based mainly on the satellite visibility plan with the following factors taken into consideration: time required to permit safe travel between survey sites; time to set up and take down the equipment before and after the survey; time of survey; and possible time loss due to unforeseeable problems or complications. Station occupation during each session should be designed to minimize travel time in order to maximize the overall efficiency of the survey.

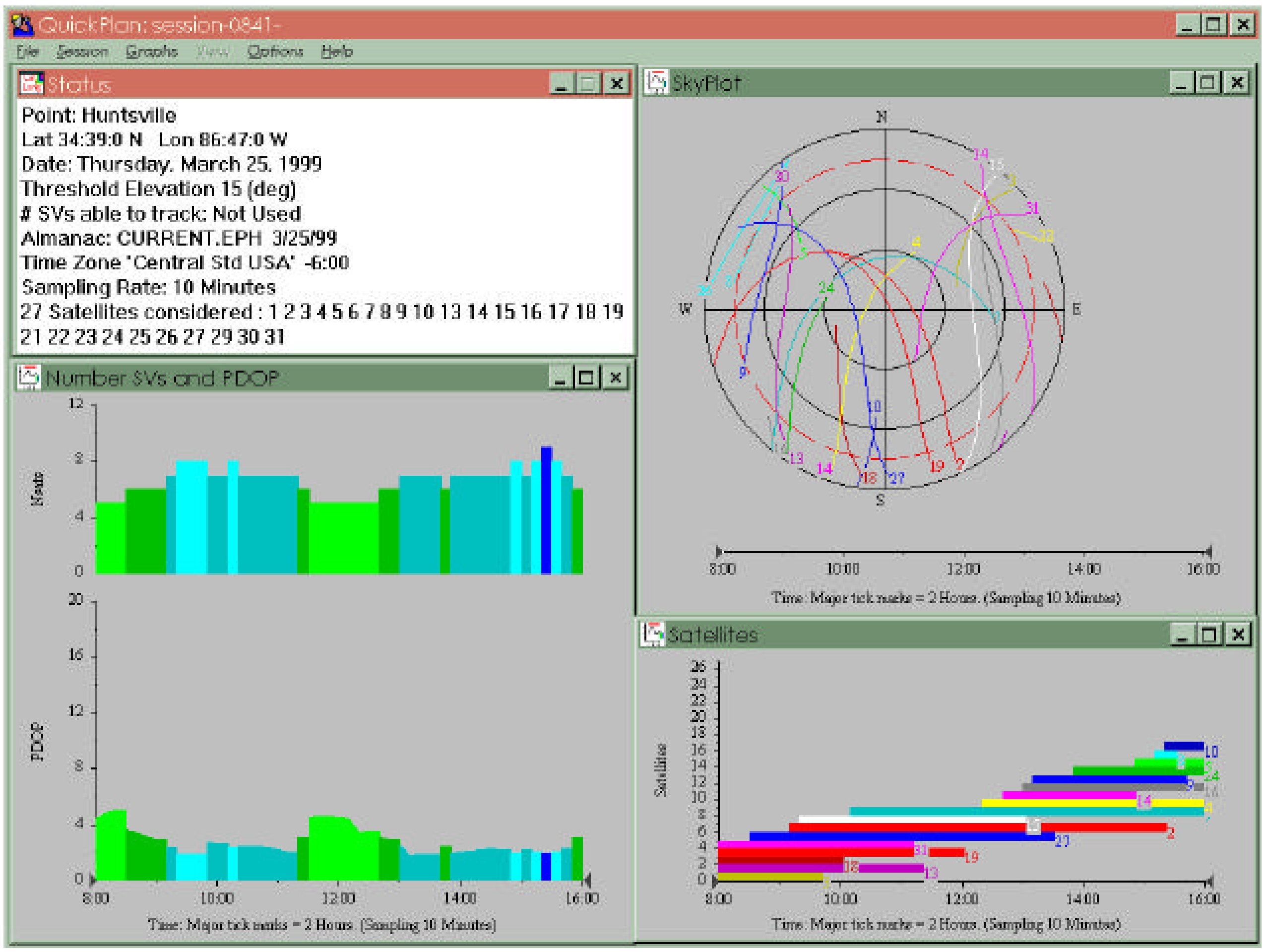


Figure 8-11. Visibility plot of satellites and PDOP versus Time

8-16. Mandatory Criteria

The guidelines in Table 8-2 and Table 8-4 shall be considered mandatory.