Chapter 3
GPS Reference Systems

3-1. General

In order to understand GPS and its positional information, it is important to understand the reference system on which it is based, and how that reference system relates to the user's local system. The GPS satellites are referenced to the World Geodetic System of 1984 (WGS 84) ellipsoid. For surveying purposes, this earth-centered WGS 84 coordinate system must be converted (i.e. transformed) to a user-defined ellipsoid and datum, such as the North American Datum of 1983 (NAD 83), NAD 27, the North American Vertical Datum of 1988 (NAVD 88), or the National Geodetic Vertical Datum of 1929 (NGVD 29). Differential positioning partially provides this transformation by locating one of the receivers at a known point on the user's reference datum or frame. However, for more precise applications, the reference datum cannot be considered as absolutely rigid in time. This chapter summarizes reference systems and datums to which GPS coordinates can be transformed.

3-2. Geodetic Coordinate Systems

The absolute positions obtained directly from GPS pseudorange measurements are based on the 3-D, earth-centered WGS 84 ellipsoid (Figure 3-1). Coordinate outputs are on a Cartesian system (X-Y-Z) relative to an Earth-Centered Earth-Fixed (ECEF) rectangular coordinate system having the same origin as the WGS 84 ellipsoid, i.e. geocentric. This geocentric X-Y-Z coordinate system should not be confused with the X-Y plane coordinates established on local grids; local systems usually have entirely different definitions, origins, and orientations which require certain transformations to be performed. WGS 84 geocentric X-Y-Z Cartesian coordinates can easily be converted into WGS 84 ellipsoid coordinates (i.e. f, l, and h--geodetic latitude, longitude, and ellipsoidal height, respectively). GPS baseline distances are computed on the geocentric coordinate system, not ellipsoidal coordinates. It is critical to note that the WGS 84 ellipsoidal height (h) is not the orthometric elevation used for civil works projects. Performing these transformations (also known as "site calibrations") from WGS 84 to local reference systems is a critical, and sometimes complicated, part of GPS surveying.

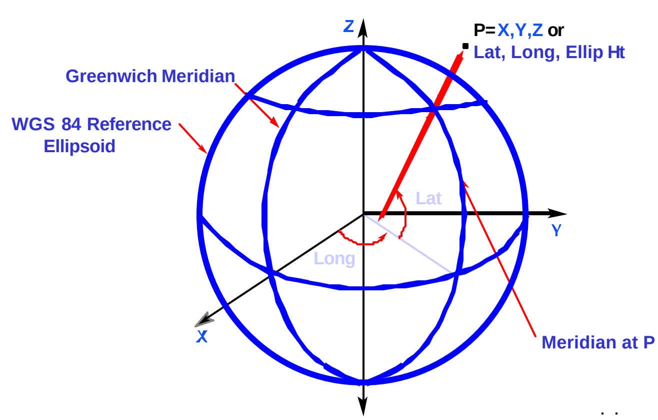

Figure 3-1. WGS 84 reference ellipsoid

3-3. WGS 84 Reference Ellipsoid

a. The origin of the WGS 84 Cartesian system is the earth's center of mass. The Z-axis is parallel to the direction of the Conventional Terrestrial Pole (CTP) for polar motion, as defined by the Bureau International Heure (BIH), and equal to the rotation axis of the WGS 84 ellipsoid. The X-axis is the intersection of the WGS 84 reference meridian plane and the CTP's equator, the reference meridian being parallel to the zero meridian defined by the BIH and equal to the X-axis of the WGS 84 ellipsoid. The Y-axis completes a right-handed, earth-centered, earth-fixed orthogonal coordinate system, measured in the plane of the CTP equator 90 degrees east of the X-axis and equal to the Y-axis of the WGS 84 ellipsoid. This system is illustrated in Figure 3-1 above. The DoD continuously monitors the origin, scale, and orientation of the WGS 84 reference frame and references satellite orbit coordinates to this frame. Updates are shown as WGS 84 (GXXX), where "XXX" refers to a GPS week number starting on 29 September 1996.

b. Prior to the development of WGS 84, there were several reference ellipsoids and interrelated coordinate systems (datums) that were used by the surveying and mapping community. Table 3-1 lists just a few of these reference systems along with their mathematical defining parameters. Transformation techniques are used to convert between different datums and coordinate systems. Most GPS software has built in transformation algorithms for the more common datums.

Table 3-1. Reference Ellipsoids and Related Coordinate Systems

Reference Ellipsoid	Coordinate System (Datum/Frame)	Semimajor axis (meters)	Shape (1/flattening)
Clarke 1866	NAD 27	6378206.4	1/294.9786982
WGS 72	WGS 72	6378135	1/298.26
GRS 80	NAD 83 (XX)	6378137	1/298.257222101
WGS 84	WGS 84 (GXXX)	6378137	1/298.257223563
ITRS	ITRF (XX)	6378136.49	1/298.25645

3-4. Horizontal Datums and Reference Frames

A major USACE application of differential GPS surveying is densifying military construction and civil works project control. This densification is usually done relative to an existing horizontal datum (NAD 27, NAD 83, or local). Even though GPS measurements are made relative to the WGS 84 ellipsoidal coordinate system, coordinate differences (i.e. baseline vectors) on this system can, for practical engineering purposes, be used directly on any local user datum. Thus, a GPS-coordinated WGS 84 baseline can be directly used on an NAD 27, NAD 83, or even a local project datum. Minor variations between these datums will be minimal when GPS data are adjusted to fit between local datum stations. Such assumptions may not be valid when high-order National Geodetic Reference System (NGRS) network densification work is being performed or where coordinates are developed relative to distant reference stations. The following paragraphs describe some of the reference systems used by the Corps for military construction and civil works projects. Much of it is extracted from *Modern Terrestrial Reference Systems*, (Snay & Soler 1999). Far more detailed descriptions of these reference systems can be found in the referenced publication (*Professional Surveyor Magazine*).

a. North American Datum of 1927 (NAD 27). NAD 27 is a horizontal datum based on a comprehensive adjustment of a national network of traverse and triangulation stations. NAD 27 is a best fit for the continental United States. The fixed datum reference point is located at Meades Ranch, Kansas.

The longitude origin of NAD 27 is the Greenwich Meridian with a south azimuth orientation. The original network adjustment used 25,000 stations. The relative precision between initial point monuments of NAD 27 is by definition 1:100,000, but coordinates on any given monument in the network contain errors of varying degrees. As a result, relative accuracies between points on NAD 27 may be far less than the nominal 1:100,000. The reference units for NAD 27 are US Survey Feet. This datum is no longer supported by NGS, and USACE commands are gradually transforming their project coordinates over to the NAD 83 described below. Approximate conversions of points on NAD 27 to NAD 83 may be performed using CORPSCON, a transformation program developed by ERDC/TEC--see EM 1110-1-1004. Since NAD 27 contains errors approaching 10 m, transforming highly accurate GPS observations to this antiquated reference system is not the best approach.

b. North American Datum of 1983 (NAD 83). The nationwide horizontal reference network was redefined in 1983 and readjusted in 1986 by the National Geodetic Survey. It is known as the North American Datum of 1983, adjustment of 1986, and is referred to as NAD 83 (1986). NAD 83 used far more stations (250,000) and observations than NAD 27, including a few satellite-derived coordinates, to readjust the national network. The longitude origin of NAD 83 is the Greenwich Meridian with a north azimuth orientation. The fixed adjustment of NAD 83 (1986) has an average precision of 1:300,000. NAD 83 is based upon the Geodetic Reference System of 1980 (GRS 80), an earth-centered reference ellipsoid which for most (but not all) practical purposes is equivalent to WGS 84. With increasingly more accurate uses of GPS, the errors and misalignments in NAD 83 (1986) became more obvious (they approached 1 meter), and subsequent refinements outlined below have been made to correct these inconsistencies.

c. High Accuracy Reference Networks (HARN). Within a few years after 1986, more refined GPS measurements had allowed geodesists to locate the earth's center of mass with a precision of a few centimeters. In doing so, these technologies revealed that the center of mass that was adopted for NAD 83 (1986) is displaced by about 2 m from the true geocenter. Similarly, it was found that the orientation of the NAD 83 (1986) Cartesian axes is misaligned by over 0.03 arc seconds relative to their true orientation, and that the NAD 83 (1986) scale differs by about 0.0871 ppm from the true definition of a meter. These discrepancies caused significant concern as the use of highly accurate GPS measurements proliferated. Starting with Tennessee in 1989, each state--in collaboration with NGS and various other institutions--used GPS technology to establish regional reference frames that were to be consistent with NAD 83. The corresponding networks of GPS control points were originally called High Precision Geodetic Networks (HPGN). Currently, they are referred to as High Accuracy Reference Networks (HARN). This latter name reflects the fact that relative accuracies among HARN control points are better than 1 ppm, whereas relative accuracies among pre-existing control points were nominally only 10 ppm. The NGS opted to introduce a new scale that would be consistent with the scale of the then current global reference system known as the International Terrestrial Reference Frame of 1989 (ITRF 89). The ITRF 89 scale was based on a combination of GPS, Very Long Baseline Interferometry (VLBI), and Lunar-Laser-Ranging (LLR) measurements. The resulting scale change, equal to -0.0871 ppm, altered existing NAD 83 latitudes and longitudes insignificantly, but it systematically decreased all ellipsoidal heights by about 0.6 m. Nevertheless, this change to a more accurate scale facilitated the migration toward using GPS technology for deriving accurate orthometric heights. Positional differences between NAD 83 (1986) and NAD 83 (HARN) can approach 1 meter.

d. Continuously Operating Reference Stations (CORS). The regional HARNs were subsequently further refined (or "realized") by NGS into a network of Continuously Operating Reference Stations, or CORS. This CORS network was additionally incorporated with the International Terrestrial Reference System (ITRS), i.e. the ITRF. CORS are located at fixed points throughout CONUS and at some OCONUS points--see Figure 3-2. This network of high-accuracy points can provide GPS users with centimeter level accuracy where adequate CORS coverage exists. Coordinates of CORS stations are designated by the year of the reference frame, e.g., NAD 83 (CORS 96). Positional differences between NAD 83 (HARN) and NAD 83 (CORS) are less than 10 cm. More importantly, relative positional differences between two NAD 83 (CORSxx) points is typically less than 2 cm. Thus, GPS connections to CORS stations will be of the highest order of accuracy. USACE commands can easily connect and adjust GPS-observed points directly with CORS stations using a number of methods, including the NGS on-line program OPUS (On-Line Positioning User Service), which is described more completely in Chapter 10. OPUS provides centimeter-level adjustment connections with three nearby CORS stations, and outputs adjusted coordinates in the latest epochs of NAD 83 and ITRF systems.

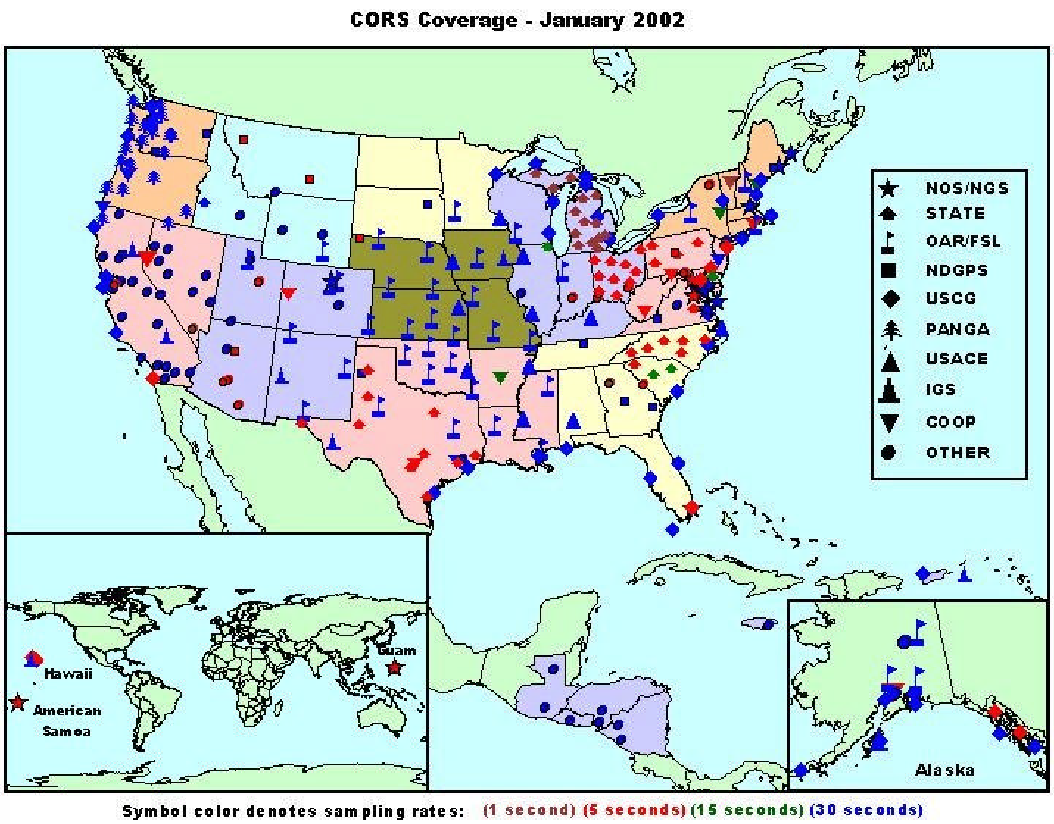

Figure 3-2. Continuously Operating Reference Stations (NGS)

e. International Terrestrial Reference Frame (ITRF). The ITRF is a highly accurate geocentric reference frame with an origin at the center of the earth's mass. The ITRF is continuously monitored and updated by the International Earth Rotation Service (IERS) using very-long-baseline-interferometry (VLBI) and other techniques. These observations allow for the determination of small movements of fixed points on the earth's surface due to crustal motion, rotational variances, tectonic plate movement, etc. These movements can average 10 to 20 mm/year in CONUS (see Figure 3-3 below), and may become significant when geodetic control is established from remote reference stations. These refinements can be used to more accurately determine GPS positions observed on the basic WGS 84 reference frame. NAD 83 coordinates are defined based on the ITRF year/epoch in which it is defined, e.g., ITRF 89, ITRF 96, ITRF 2000. For highly accurate positioning where plate velocities may be significant, users should use the same coordinate reference frame and epoch for both the satellite orbits and the terrestrial reference frame. USACE requirements for these precisions would be rare.

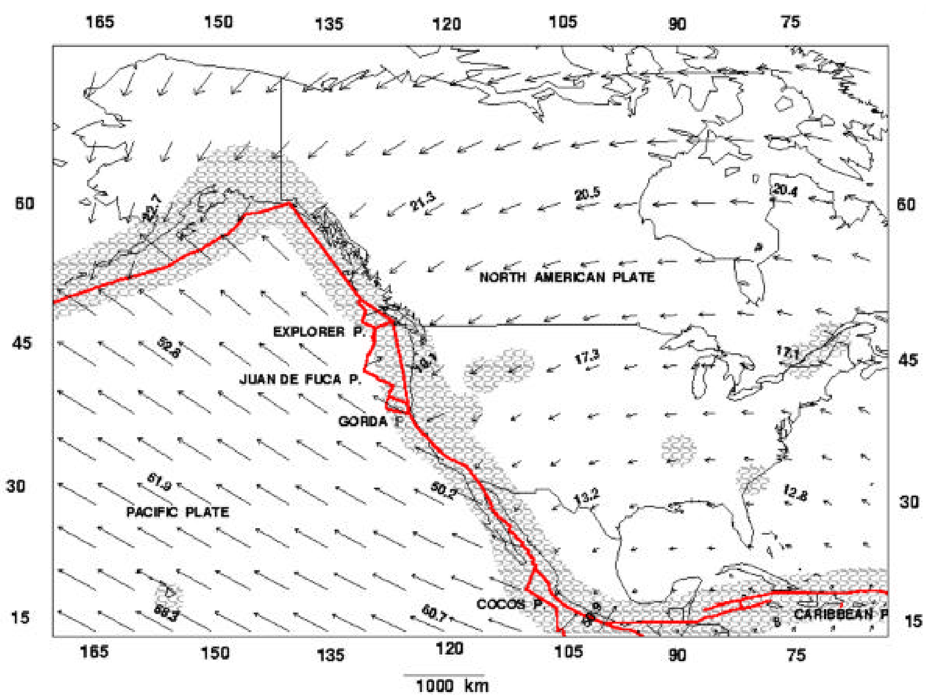

Figure 3-3. ITRF horizontal velocities in mm/year (NGS)

3-5. Transforming between Horizontal Survey Datums

Differential GPS observations routinely provide horizontal baseline accuracies on the order of 1 ppm. This far exceeds the stated 1:300,000 accuracy for NAD 83 and (approximately) 1:100,000 for NAD 27. Distortions in NAD 27 can be as much as 10 m, up to 1 m in NAD 83 (1986), and a few centimeters in NAD 83 (HARN) points. Thus, approximate transformations (e.g., CORPSCON) will retain the original distortions in the networks. Even though GPS has such a high degree of precision, it provides only coordinate differences; therefore, ties to the national network to obtain coordinates of all GPS stations must be done without distorting the established project control network (i.e. the GPS-derived vectors are "degraded" during the adjustment to "fit" the local network). Generally, on mid-size survey projects, three or more horizontal control stations from the national network can be used during the GPS observation scheme. Direct connections to CORS stations can also be made in order to update a project's control scheme to the National network. These highly accurate CORS connections will often be more accurate than the original project control scheme, and can be referenced to the latest NGS NAD/ITRF time-dependent reference frame if needed. NGS has developed a software package that provides time-related transformations between the varied NAD 83 and ITRF reference frames--"Horizontal Time-Dependent Positioning" or HTDP. This software transforms positions and velocities between ITRF xx, WGS 84 (Gxxx), and NAD 83. It updates and predicts displacements between dates (epochs) due to crustal motion. In order to facilitate a tie between GPS and existing networks for horizontal control, a re-adjustment of the whole local project network scheme (all control and GPS-derived points) should be performed. There are many commercial software packages that can be used to perform this adjustment.

Once a network adjustment meets the accuracy requirement, those values should not be readjusted with additional points or observations unless a complete readjustment is performed.

a. Transforming to local project datums. Corps construction and navigation projects are often based on local project datums. Usually, but not always, these local station-offset datums are referenced to the NGRS. They may also have SPCS coordinates of uncertain origin. These local datums might be accurate within a small area, but can become distorted over larger areas. When local project datums are not connected to any regional datum, GPS ties can be observed to outside control in order to transform the local datum to an established reference datum. GPS receiver vendors usually provide software transformation options for converting WGS 84 coordinates to local datums. For small survey areas, a Three-Parameter Transformation is adequate. For larger areas, a Seven-Parameter Transformation should be performed. In addition, local horizontal control coordinates must be "calibrated" to the WGS 84 scheme used by GPS. This is termed "site calibration" in Trimble software. Although only two points are required, at least three established control points should be occupied (and connecting baselines observed) to perform a datum transformation. These observations provide horizontal translation, rotation, and scale parameters between the two grids. Thus, with these datum translations and calibrations, observed GPS data is "best-fit" onto the local grid system. Many least squares adjustment packages also contain datum transformation routines that can be used to convert local datums to regional networks.

b. State Plane Coordinate System (SPCS). The State Plane Coordinate System (SPCS) was developed by the NGS to provide a planar representation of the earth's surface. Most USACE civil and military projects require project coordinates in a SPCS format, or occasionally in the military Universal Transverse Mercator (UTM) plane coordinate system. To properly relate spherical coordinates (f, l) to a planar system (Northings and Eastings), a developable surface must be constructed. A developable surface is defined as a surface the can be expanded without stretching or tearing. The two most common developable surfaces or map projections used in surveying and mapping are the cone and cylinder. The projection of choice is dependent on the north-south or east-west extent of the region. Areas with limited east-west dimensions and elongated north-south extent utilize the cylindrical Transverse Mercator projection. Areas with limited north-south dimensions and elongated east-west extent utilize the conical Lambert projection. SPCS are different for the NAD 27 and NAD 83 reference systems. Figure 3-4 below shows the layout for the various SPCS (NAD 83) zones. For further information on the State Plane Coordinate System see EM 1110-1-1004.

Figure 3-4. State plane coordinate zones (NAD 83)

c. Practical considerations in USACE. Few, if any, USACE civil works and military construction projects require high-precision geodetic control referenced to the latest ITRF time epoch to account for polar motion, tectonic plate movement, etc. These refinements require additional observation and office adjustment and analysis time, and can significantly increase project costs. Requirements for high precision geodetic control are a function of project function and size. For example, a major watershed with significant hydraulic complications may require high-accuracy CORS connections for vertical control purposes. However, a small, shallow-draft navigation project that is dredged once every 3 years would not need these high-order framework references. In addition, repeated transforms and readjustments of project control can result in mixed reference schemes, and can cause construction claims. This may occur if subsequent users performing topographic or GIS mapping use superseded coordinate systems. Thus, project managers and surveyors need to consider the project function and future developments in determining the framework accuracy.

3-6. Orthometric Elevations

Orthometric elevations are those corresponding to the earth's irregular geoidal surface, as illustrated in Figure 3-5 below. Measured differences in elevation from spirit leveling are generally relative to the local geoidal surface--a spirit level bubble (or pendulum) positions the instrument normal to the direction of gravity, and thus parallel with the local slope of the geoid. The orthometric height of a point is the distance from the geoid (or a related reference surface) to the point on the earth's surface, measured along the line perpendicular to every equipotential surface in between. A series of equipotential surfaces can be used to represent the gravity field. One of these surfaces, the geoid, is specified as the reference system from which orthometric heights are measured. The geoid itself is defined as an equipotential surface. Natural variations in gravity induce a smooth, continuous, curvature to the plumb line, and therefore physical equipotential surfaces which are normal to gravity do not remain geometrically parallel over a given vertical distance (i.e. the plumb line is not quite parallel to the ellipsoidal normal). Elevation differences between two points are orthometric differences, a distinction particularly important in river/channel hydraulics. Orthometric heights for the continental United States (CONUS) are generally referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29) or the updated North American Vertical Datum of 1988 (NAVD 88); however, other vertical datums may be used in some projects (e.g., the International Great Lakes Datum of 1955 (IGLD 55) and the revised International Great Lakes Datum of 1985 (IGLD 85). Hydraulic-based "dynamic" elevation datums, such as IGLD, differ from orthometric elevations in that elevation differences are based on hydraulic head (i.e. work) differences. The NGVD 29 reference datum approximates mean sea level--the NAVD 88 does not. Tidal reference datums (e.g., MLLW) vary geographically over short distances and must be accurately related to NAVD 88 and/or NGVD 29 orthometric heights. River systems may have local flow/discharge referenced datums--see EM 1110-2-1003. GPS derived ellipsoidal heights shown in Figure 3-5 below must be converted to local orthometric elevations in order to have useful engineering and construction value. This transformation is usually done by a form of "site calibration" using known orthometric elevations of fixed benchmarks and/or geoid undulation models for the project area. These transforms are further explained below.

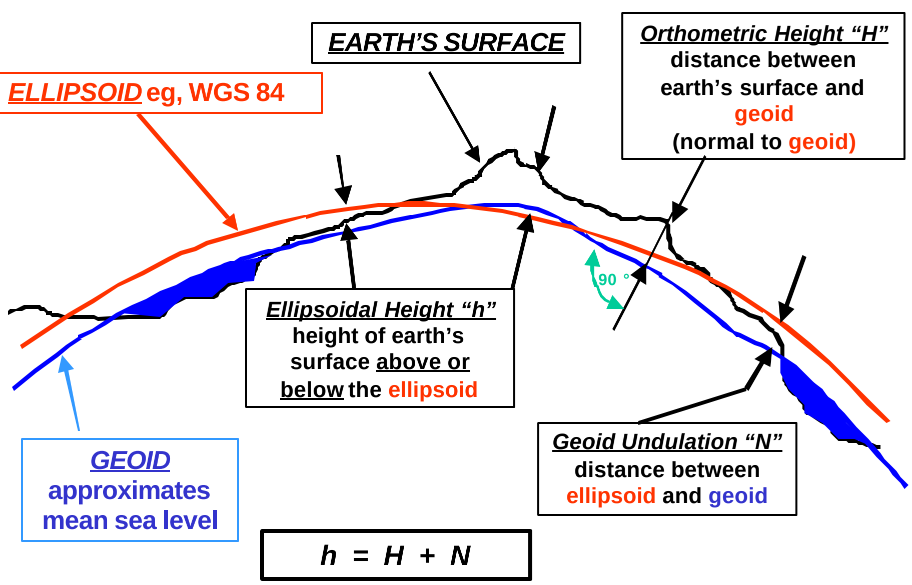

Figure 3-5. Ellipsoid, geoid, and earth's surface definitions and relationships

3-7. WGS 84 Ellipsoidal Heights

In the US, final positions from DGPS are established with respect to NAD 83. Since NAD 83 is based on the GRS 80 ellipsoid, ellipsoid heights obtained from GPS surveying using NAD 83 control are based on the GRS 80 ellipsoid. These heights are referred to as NAD 83 GPS ellipsoidal heights. Unlike the WGS 84 ellipsoid, the GRS 80 ellipsoid is not exactly geocentric, which can create problems (i.e. large errors) when converting NAD 83 GPS ellipsoid heights to orthometric heights using some geoid models. GPS-determined heights (or height differences) are referenced to an idealized mathematical ellipsoid that differs significantly from the geoid; thus, GPS heights are not the same as the orthometric heights needed for standard USACE projects (i.e. local engineering, construction, and hydraulic measurement functions). Accordingly, any WGS 84 referenced ellipsoidal height obtained using GPS must be transformed or calibrated to the local orthometric vertical datum. This requires adjusting and interpolating GPS-derived heights relative to fixed orthometric elevations. Over short distances--less than 1 km--elevation differences determined by GPS can usually be assumed to be orthometric differences. These elevation differences would then be of sufficient accuracy for topographic site plan mapping, such as those acquired using RTK total station methods. However, when GPS is used to establish primary vertical control benchmarks for a project, special procedures and cautions must be observed, e.g., measurements should be made relative to higher-order NGRS benchmarks in order to develop the best model for a project. Such a process may or may not be of suitable accuracy (i.e. reliability) for establishing primary control on some engineering and construction work--see Chapter 8.

3-8. Orthometric Height and WGS 84 Ellipsoidal Elevation Relationship

Geoidal heights represent the geoid-ellipsoid separation distance measured along the ellipsoid normal and are obtained by taking the difference between ellipsoidal and orthometric height values. Knowledge of the geoid height enables the evaluation of vertical positions in either the geodetic (ellipsoid based) or the orthometric height system. The relationship between a WGS 84 ellipsoidal height and an orthometric height relative to the geoid can be obtained from the following equation, as depicted graphically in Figure 3-5 above.

$$h = H + N \qquad \text{(Eq 3-1)}$$

where

h = ellipsoidal height (WGS 84)
H = elevation (orthometric--normal to geoid)
N = geoidal undulation above or below the WGS 84 ellipsoid

and by convention the geoid undulation *"N"* being a positive height when above the ellipsoid.

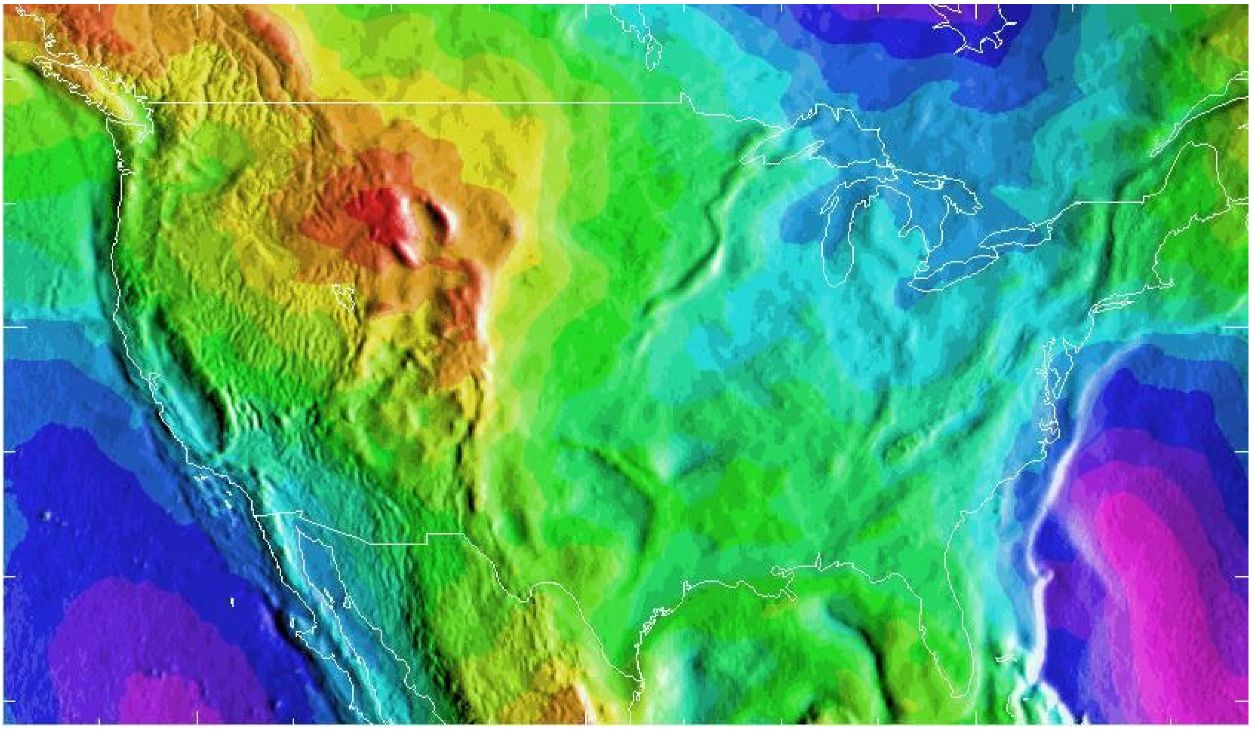

Figure 3-6. Geoid undulation model of North America--depicts geoid undulation "*N*" relative to the WGS 84 ellipsoid

3-9. Geoid Undulations and Geoid Models

Due to significant variations in the geoid, sometimes even over small distances, elevation differences obtained by GPS cannot be directly equated to orthometric (or spirit level) differences. This geoid variation is depicted as a surface model in Figure 3-6 above. Geoid modeling techniques are used to obtain the parameter *"N"* in Equation 3-1, from which ellipsoidal heights can be converted to orthometric elevations. These geoid models (e.g., Geoid 90, Geoid 93, Geoid 96, Geoid 99, etc.) are approximations based on observations by the NGS. Each successive geoid model is more accurate. In time, these models may improve to centimeter-level accuracy. On some small project areas where the geoid stays fairly constant, elevation differences obtained by GPS can be directly used without geoid correction. Geoid models are not compatible with the superseded NGVD 29.

a. Geoid height values at stations where either only " h "or " H " is known can be obtained from geoid models that are mathematical surfaces representing the shape of the earth's gravity field. The geoid model is constructed from a truncated functional series approximation using a spherical harmonics expansion and an extensive set of globally available gravity data. The model is determined from the unique coefficients of the finite series representing the geoid surface. Its accuracy depends on the coverage and accuracy of the gravity measurements used as boundary conditions. Former geoid models produced for general use limit absolute accuracies for geoid heights to no less than 1 meter. More recent geoid models have shown a significant increase in geoid height absolute accuracy--down to the centimeter level.

b. In practice, the shape of the geoid surface is estimated globally as a function of horizontal coordinates referenced to a common geocentric position. Specific geoid height values are extracted from the model surface at the node points of a regular grid (i.e. a 2-minute x 2-minute grid spacing). Biquadratic interpolation procedures can be used within a grid cell boundary to approximate the geoid height at a given geodetic latitude and longitude. For example, the NGS GEOID 96 model for the United States indicates geoid heights (*N*) range from a low of (-) 51.6 meters in the Atlantic to a high of (-) 7.2 meters in the Rocky Mountains. For more information on geoid modeling, see the references listed in Appendix A.

c. GPS surveys can be designed to provide elevations of points on any local vertical datum. This requires connecting to a sufficient number of existing orthometric benchmarks from which the elevations of unknown points can be "best-fit" or "site calibrated" by some adjustment method--usually a least-squares minimization. This is essentially an interpolation process and assumes linearity in the geoid slope between two established benchmarks. If the geoid variation is not linear--as is typically the case--then the adjusted (interpolated) elevation of an intermediate point will be in error. Depending on the station spacing, location, local geoid undulations, and numerous other factors, the resultant interpolated/adjusted elevation accuracy is usually not suitable for construction surveying purposes; however, GPS-derived elevations may be adequate for small-scale topographic mapping control.

3-10. North American Vertical Datum of 1988 (NAVD 88)

The NAVD 88 datum is the product of a vertical adjustment of leveled height difference measurements made across North America. This reference system supersedes the NGVD 29 vertical reference framework. NAVD 88 was constrained by holding fixed the orthometric height of a single primary tidal benchmark at Father's Point / Rimouski, Quebec, Canada and performing a minimally constrained general adjustment of US-Canadian-Mexican leveling observations. The vertical reference surface is therefore defined by the surface on which the gravity values are equal to the control point value. NAVD 88 elevations are published orthometric heights that represent the geometric distance from the geoid to the terrain measured along the plumb line. Orthometric height corrections were used to enforce consistency between geopotential based vertical coordinates and measured leveled differences. NAVD 88 is the most compatible vertical reference frame available to relate GPS ellipsoidal heights to orthometric heights. Note also that NGVD 29 is no longer supported by NGS; thus, USACE commands should be transitioning all older project vertical control to NAVD 88. The differences in orthometric elevations between the superseded NGVD 29 and NAVD 88 references are significant--upwards of 1.5 meters in places, as depicted in Figure 3-7 below. Therefore, it is important that these two reference systems not be confused. Given the local variations shown in Figure 3-7, there is no direct transformation between the two systems, and a site calibration/transformation must be performed as explained in subsequent sections.

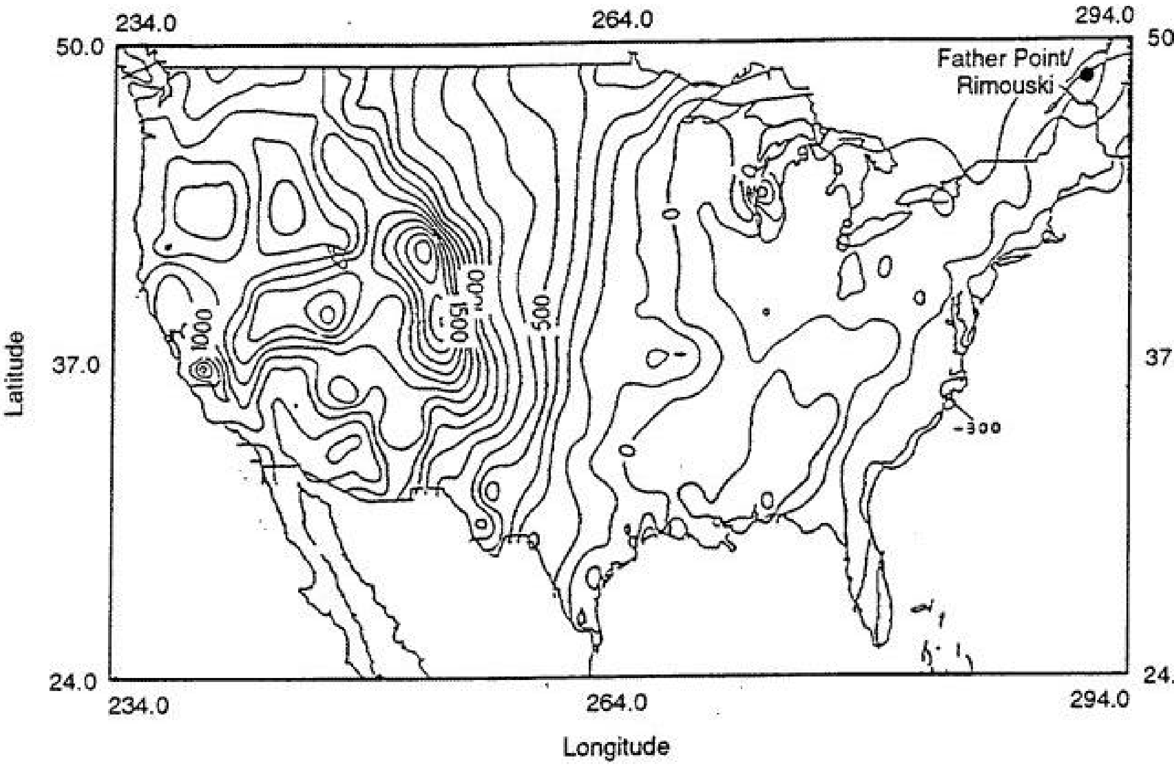

Figure 3-7. NGVD 29-NAVD 88 elevation differences in mm

3-11. Using GPS to Densify Orthometric Elevations

DGPS observation sessions produce 3-D geodetic coordinate differences that establish the baseline between two given stations. The expected accuracy of ellipsoidal height difference measurements is based on several factors, such as GPS receiver manufacture type, observation session duration, and the measured baseline distance, but it does not depend greatly on prior knowledge of the absolute vertical position of either occupied station. Dual-frequency, carrier phase measurement based GPS surveys are usually able to produce 3-D relative positioning accuracies under 30 mm at the 95% confidence level over baseline distances less than 20 km, depending on the type of GPS surveying method used. This situation exists mainly because GPS range biases are physically well correlated over relatively short distances and tend to cancel out as a result of forming double differences for carrier phase data processing. In contrast, GPS absolute code positioning accuracy will contain the full effects of any GPS range measurement errors. Geoidal height differences describe the change in vertical position of the geoid with respect to the ellipsoid between two stations. These relative geoidal heights can be more accurate than the modeled absolute separation values within extended areas because the relative geoidal height accuracy is based on the continuous surface characteristics of the geoid model, where only small deviations between closely spaced points would be expected. The regional trend or slope of the geoid at a given point will not be highly sensitive to local gravity anomalies especially in non-mountainous areas. Differential GPS can fairly accurately measure ellipsoidal height differences from GPS satellites. GPS surveys output vertical positions in geodetic coordinates defined with respect to the WGS 84 reference ellipsoid. The ellipsoidal height value at a given point is based on the distance measured along the normal vector from the surface of the reference ellipsoid to the point. The practical accuracy of WGS 84 as a vertical reference frame for collecting elevation data depends on the actual ellipsoidal height values assigned to benchmarks or other physically defined control points.

3-12. GPS Vertical Site Calibration

A calibration is needed in real-time surveying in order to relate GPS positions that are measured in terms of WGS-84 to local grid coordinate projections, such as SPCS, UTM, or a local station-offset-elevation system. In addition, a vertical calibration is needed to adjust the observed GPS ellipsoid elevations to a local vertical datum, and account for undulations in the local geoid over the project area. A calibration should be used on a project whenever new points are to be established. A calibration is based on a set of points that have 3-D coordinates in both WGS-84 and the local grid coordinate projection system. The quality of the calibration will be affected by the accuracy and consistency of the GPS coordinates of the points. Points tied to the NGRS are recommended as the basis of a calibration. The number of points that can be used in a calibration is manufacturer and software dependent. Smaller sized projects may be calibrated with one 3-D point. However, for larger sized projects, three or four 3-D points are recommended. Calibration points should be well distributed around the project exterior. Projects may be calibrated by two methods: (1) in the field in the survey data collector or (2) in the network adjustment. The latter procedure is recommended for large projects. The calibration computation summary should be examined for reasonable results in the horizontal scale, maximum vertical adjustment inclination, and the maximum horizontal and vertical residuals.

a. Figure 3-8 below illustrates the varied requirements for vertical site calibrations. This figure depicts a typical contour plot of a geoid model--height differences between the geoid relative to the WGS 84 ellipsoid. In the large (8 km x 8 km) Area A, the geoid undulation varies from 0.80 to 1.27 m--nearly a 50 cm variation. In order to determine accurate orthometric elevations from GPS ellipsoid elevation observations, this variation in the geoid must be accurately accounted for. In addition, the published orthometric elevations at each of the 7 established control benchmarks may not fit exactly with the geoid model--the geoid model may have been approximated from other NGRS points. Therefore, GPS observations over the 7 established control network points must be adjusted to further refine the geoid model so that subsequent GPS observations to any point in the project area can be "best-fitted" to the local vertical datum. Solely relying on a published geoid model is not recommended--connections with existing control should always be observed to refine the model. GPS adjustment software must be able to compensate for both the variations in the geoid model and variations in the established control benchmarks. In order to accomplish this, GPS observations need to be connected between the fixed control benchmarks, as shown in Area A.

b. The small (1 km x 1 km) Area B in Figure 3-8 is more typical of local RTK topographic survey projects. The geoid model shows a minimal undulation over this area--from 0.72 m to 0.75 m. This 3 cm variation may or may not be significant, depending on the required elevation accuracy of the survey. If this 3 cm geoid variation is not considered significant, then the geoid undulation at the selected reference station could be used over the entire area, and no geoid model correction used. Alternatively, the 2 control benchmarks could be calibrated and the geoid model included in the adjustment. When 2 control benchmarks are available, as shown around Area B, then a GPS check between the benchmarks is recommended. If the geoid model is not used, the geoid correction could be interpolated from the check baseline observation results, holding the 2 control points fixed.

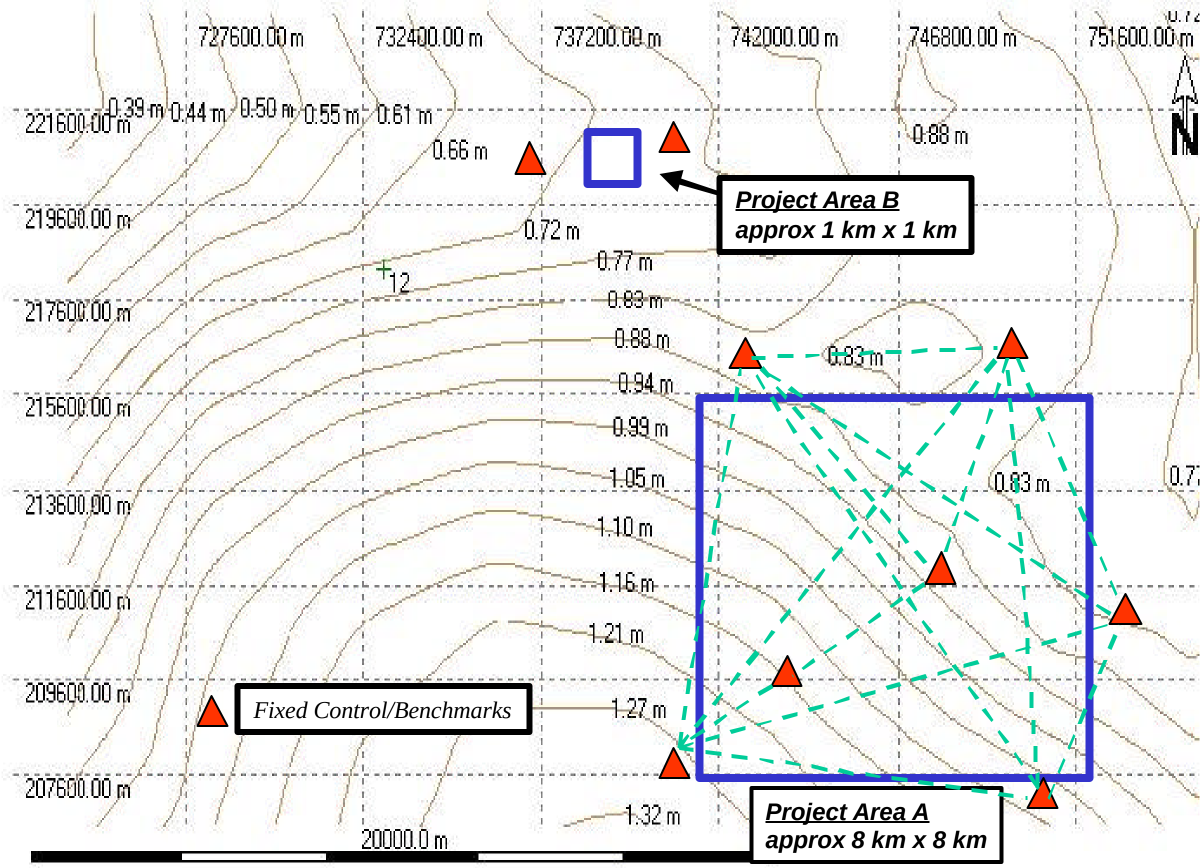

Figure 3-8. Plot of geoid undulation contours over a local survey area (Leica)

c. Figure 3-9 below illustrates vertical calibrations over small local survey areas, which is typical of Corps topographic survey applications. This area contains two fixed benchmarks with local datum elevations. A GPS reference receiver is set up over one benchmark and baseline hubs are staked out relative to this point, using kinematic techniques. The second fixed benchmark is used as a check point. A local geoid model shows estimated geoid heights varying between -11.23 and -11.25 m. Orthometric elevations on the individual baseline hubs are computed by correcting the observed ellipsoidal elevation differences with the local geoid undulation differences. This local geoid elevation difference (- 2 cm) could have been ignored if this error is acceptable to project accuracy requirements. This would, in effect, assume observed ellipsoidal elevation differences are equal to orthometric elevation differences and no geoid model corrections are applied to the observations.

d. In Figure 3-9, a check point GPS elevation difference of +12.40 m is observed. The published orthometric elevation difference between these points is +12.42 m. This confirms the geoid model is accurate over this area since the computed geoid undulation difference (ΔN) is - 0.02 m (+12.40-12.42). Had a large misclosure existed at the check point, then either the published elevations are inaccurate or the geoid model is inaccurate, or both. A GPS baseline check to a third benchmark would be required, or conventional levels could be run between the two fixed points to resolve the problem.

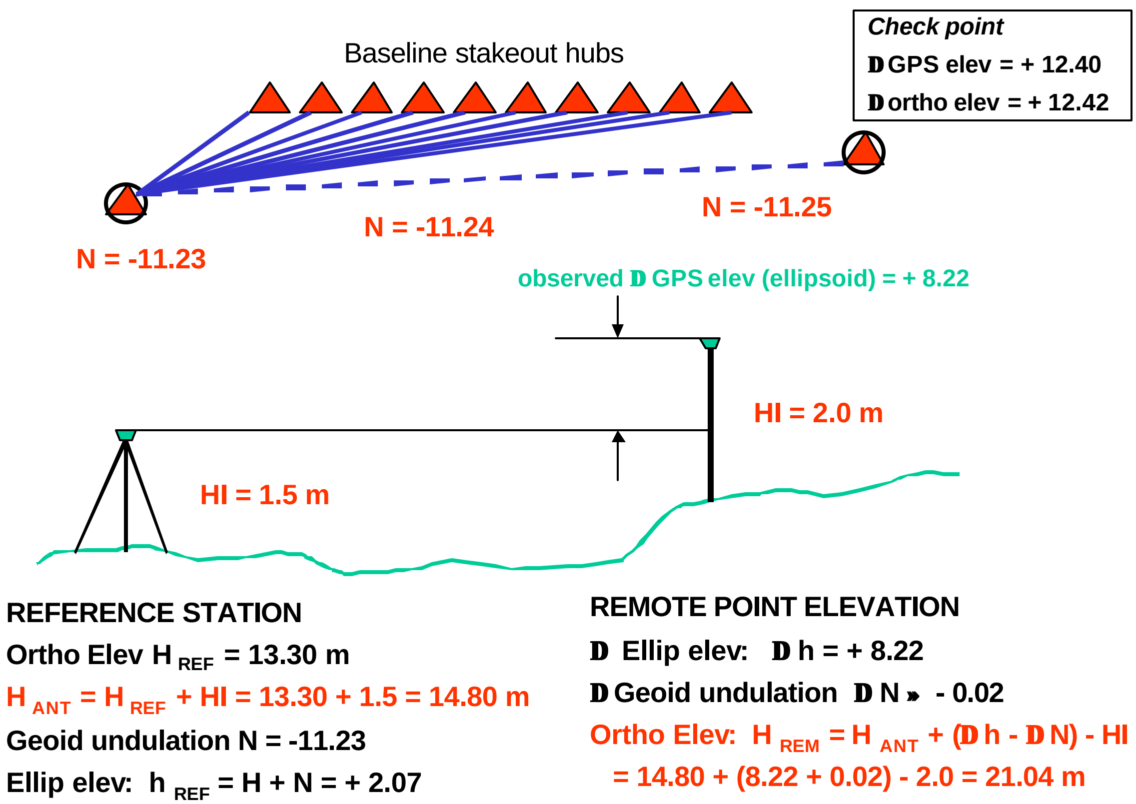

Figure 3-9. Geoid elevation corrections for localized surveys

e. For further references on GPS site calibrations, refer to Trimble's *Real-Time Surveying Workbook* (Trimble 2000b).

3-13. GPS Time References

Time used for most purposes is based on an astronomic (solar) time measure, or "universal time"--UT. UT is based on the earth's rotation. Other time references include UT 0 (which is based on astronomical observations), UT 1 (UT 0 corrected for polar motion and equals Greenwich Mean Time--GMT), and Atomic Time (AT). GPS satellites have atomic clocks which output a time base that is not related to astronomic time measures. However, these different time scales can be coordinated. GPS time is accurately maintained and monitored by the DoD. GPS time is usually maintained within 30 nanoseconds of Universal Coordinated Time (US Naval Observatory), or UTC (USNO). GPS time is based on a reference "GPS epoch" of 000 hours (UTC) 6 January 1980. From DoD (1996), the relationship between GPS time and UTC is:

GPS time = UTC + number of leap seconds + [GPS-to-UTC bias]

GPS receivers obtain time corrections from the broadcast data messages and can thus output UTC (USNO) time increments. UTC is the time used for many USACE surveying applications where time is transferred from a GPS receiver in order to coordinate data streams from some peripheral sensing device--e.g., inertial measurement units (IMU), cameras, acoustic depth recorders, etc. It is especially critical to determine any latencies between the GPS satellite acquisition and the time tag of the subsequent position computation, and to correlate this time tag latency with other peripheral sensors.