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 (WGS84) ellipsoid. For surveying purposes, this earth-centered WGS84 coordinate system must be converted (i.e. transformed) to a user-defined ellipsoid and datum, such as the North American Datum of 1983 (NAD83) or the North American Vertical Datum of 1988 (NAVD88). 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 positions can be transformed. Refer to additional guidance in ER 1110-2-8160 (*Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums*) and its implementing manual EM 1110-2-6056 (*Standards and Procedures for referencing Project Elevations Grades to Nationwide Vertical Datums*)

3-2. Geodetic Coordinate Systems. The absolute positions obtained directly from GPS pseudorange measurements are based on the 3-D, earth-centered WGS84 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 WGS84 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, such as the U.S. State Plane Coordinate System and UTM. Local systems usually have entirely different definitions, origins, and orientations which require certain transformations to be performed. WGS84 geocentric X-Y-Z Cartesian coordinates can easily be converted into WGS84 ellipsoid coordinates (i.e. ϕ, λ, 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 WGS84 ellipsoidal height (*h*) is not the orthometric elevation (H) used for civil works projects. Performing these transformations (also known as "site calibrations") from WGS84 to local reference systems is a critical, and sometimes complicated, part of GPS surveying.

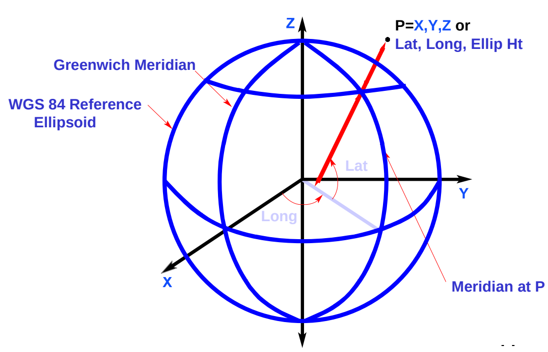

Figure 3-1. WGS84 reference ellipsoid

3-3. WGS84 Reference Ellipsoid. The origin of the WGS84 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 WGS84 ellipsoid. The X-axis is the intersection of the WGS84 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 WGS84 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 WGS84 ellipsoid. This system is illustrated in Figure 3-1. The DoD continuously monitors the origin, scale, and orientation of the WGS84 reference frame and references satellite orbit coordinates to this frame. Updates are shown as WGS84 (GXXX), where "XXX" refers to a GPS week number starting on 29 September 1996. Prior to the development of WGS84, 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	NAD27	6378206.4	1/294.9786982
WGS72	WGS72	6378135	1/298.26
GRS80	NAD83 (XX)	6378137	1/298.257222101
WGS84	WGS84(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 the densification of military construction and civil works project control. This is usually done relative to an existing horizontal datum (NAD83 or local). Even though GPS measurements are made relative to the WGS84 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 WGS84 baseline can be directly used on a NAD83 or 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 Spatial Reference System (NSRS) 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 USACE for military construction and civil works projects. Typical reference frames in use today include:

a. North American Datum of 1927 (NAD27). NAD27 is a horizontal datum based on a comprehensive adjustment of a national network of traverse and triangulation stations.

b. North American Datum of 1983 (NAD83). The nationwide horizontal reference network was redefined in 1983 and adjusted 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 NAD83 (1986).

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. Starting with Tennessee in 1989, each state--in collaboration with NGS and various other institutions--used GPS technology to establish regional adjustment/realization that were to be consistent with NAD83. A national readjustment of all state-wide HARNs has been completed and a new realization of NAD83 has been published, referred to as NAD83 (NSRS2007).

d. The NAD83(CORS96) and NAD83(2007) realizations of the NAD83. NGS has adopted a realization of NAD83 called NAD83(2007) for the distribution of coordinates at ~70,000 passive geodetic control monuments. This realization *approximates* (but is not, and can never be, equivalent to) the more rigorously defined NAD83(CORS96) realization in which Continuously Operating Reference Station (CORS) coordinates are distributed. NAD83(NSRS2007) was created by adjusting GPS data collected during various campaign-style geodetic surveys performed between the mid-1980's and 2005. For this adjustment, NAD83(CORS96) positional coordinates for ~700 CORS were held fixed (predominantly at the 2002.0 epoch for the stable north American plate, but 2007.0 in Alaska and western CONUS) to obtain consistent positional coordinates for the ~70,000 passive marks, as described by Vorhauer [2007]. Derived NAD83(2007) positional coordinates should be consistent with corresponding NAD83(CORS96) positional coordinates to within the accuracy of the GPS data used in the adjustment and the accuracy of the corrections applied to these data for systematic errors, such as refraction. In particular, there were no corrections made to the observations for vertical crustal motion when converting from the epoch of the GPS survey into the epoch of the adjustment, while the NAD83 (CORS96) coordinates do reflect motion in all three directions at CORS sites. For this reason alone, there can never be total equivalency between NAD83(2007) and NAD83(CORS96). NGS has not computed NAD83(2007) velocities for any of the ~70,000 passive marks involved in this adjustment. Also, the positional coordinates of a passive mark will make reference to an "epoch date". Epoch dates are the date for which the positional coordinates were adjusted, and are therefore considered "valid" (within the tolerance of not applying vertical crustal motion). Because a mark's positional coordinates will change due to the dynamic nature of the earth's crust, the coordinate of a mark on epochs different than the listed "epoch date" can only be accurately known if a 3-dimensional velocity has been computed and applied to that mark (see http://www.ngs.noaa.gov/NationalReadjustment/difference.html). NGS maintains an active and growing network of CORS sites (1450+ as of December 2009) throughout the United States and its territories that provide GPS carrier phase and code range measurements in support of 3-dimensional positioning activities (see Figure 3-2). The coordinates of these sites are maintained with respect to both the ITRF and NAD83. They serve as the fixed control from which other passive control points of the NSRS are computed by NGS based upon GPS observation and network adjustment.

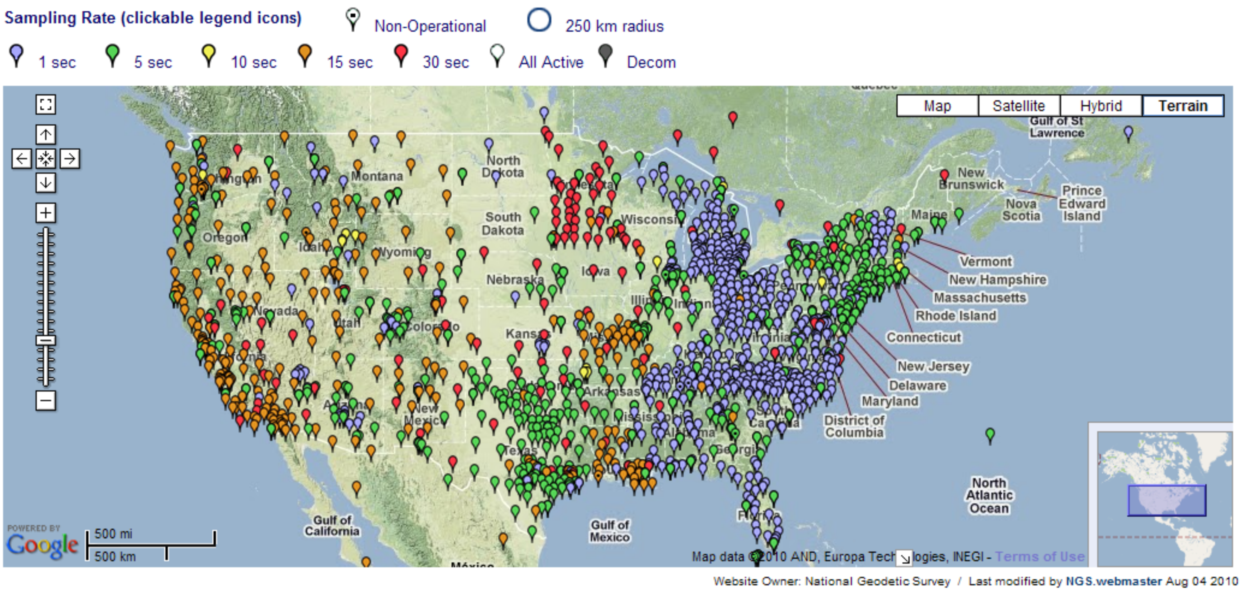

Figure 3-2. Continuously Operating Reference Stations (NGS) as of August 2010

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. 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. Additional information on the reference frames above is available in Appendix B of EM 1110-2-6056, Standards And Procedures For Referencing Project Elevation Grades To Nationwide Vertical Datums.

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 NAD83 and (approximately) 1:100,000 for NAD27. Distortions in NAD27 can be as much as 10 m, up to 1 m in NAD83 (1986), and a few centimeters in NAD83 (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 NAD83 and ITRF reference frames--"Horizontal Time-Dependent Positioning" or HTDP. This software transforms positions and velocities between ITRF xx, WGS84 (Gxxx), and NAD83. 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 NSRS. They may also have coordinates tied to a State Plane System, though the origin of these ties may not be well documented. 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 observations to outside control facilitate the transformation from a local datum to an established reference datum. For small survey areas, a Three-Parameter Transformation is adequate. For larger areas, a Seven-Parameter Transformation should be performed. In addition, the relationship of local horizontal control coordinates with respect to the NSRS must be carefully developed. 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 (ϕ,λ) 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 NAD27 and NAD83 reference systems. Figure 3-3 shows the layout for the various SPCS (NAD83) zones. For further information on the State Plane Coordinate System see Appendix B of EM 1110-2-6056, Standards And Procedures For Referencing Project Elevation Grades To Nationwide Vertical Datums.

c. Other Consideration in Regard to Localization (Site Calibration). Localization should be dealt with very carefully. A distinction should be made between localizing on published (passive) NSRS control monumentation and localizing on control monumentation and associated legacy coordinates established by the Corps or its contractors that might typically be based on conventional surveys of relatively low precision. In the former case, field localization or field calibration on valid NSRS control is recommended. In the latter, it is generally recommend that current-epoch positions of recovered Corps/contractor control monuments be recorded and later evaluated carefully against their respective legacy values in the office. Simply adopting the localization or site-calibration result, as generated in the field data collector/controller, may be risky on at least two counts. Firstly, it may mask a significant degree of non-conformality between the observed current-epoch coordinates and the legacy coordinates and introduce an undesired distortion in the recorded coordinates of other surveyed features. Secondly, the type of transformation applied (2d or 3d-conformal, affine, other) and associated parameter set may never be adequately documented and reported, and instead coordinate data may be reported/delivered as if it were current-epoch (or is assumed to be current-epoch). And once the monuments used in localization are disturbed or destroyed, and if all data and metadata relating to the localization effort are not adequately documented, confusion and costly delays may follow when an attempt is made to integrate or harmonize the localized data with other surveys based on current-epoch or other localized systems.

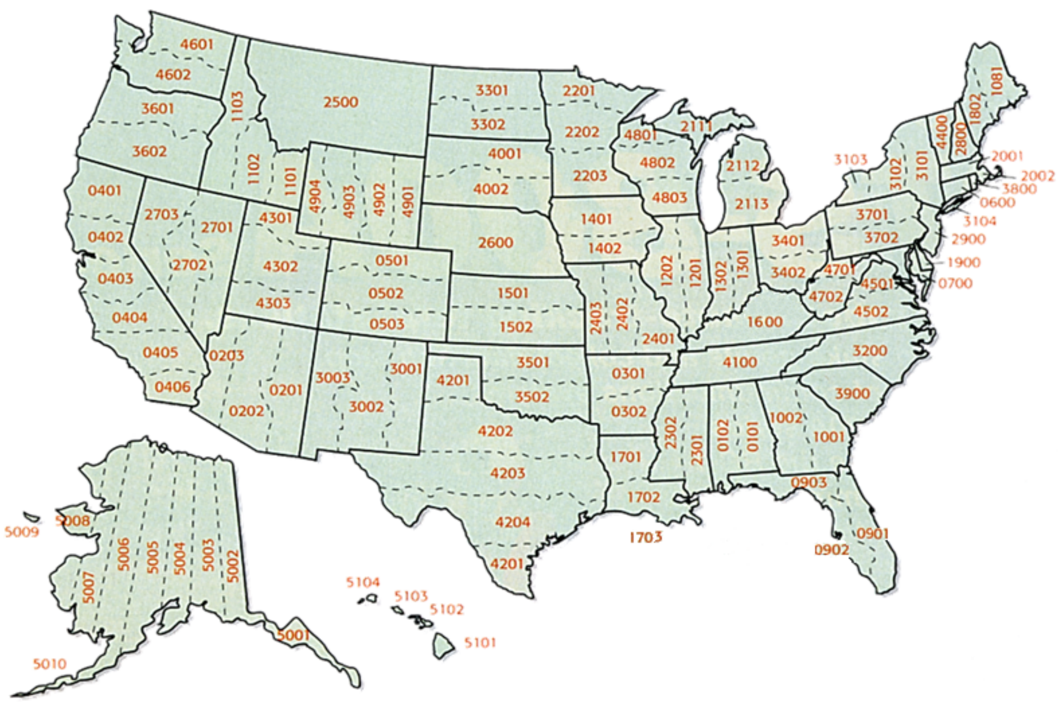

Figure 3-3. State plane coordinate zones (NAD83)

d. Practical considerations for USACE projects. Few, if any, USACE military and civil works construction projects require high-precision geodetic control referenced to the latest ITRF time epoch, accounting 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. Further guidance may be found in ER 1110-2-8160 (*Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums*) and its implementing manual EM 1110-2-6056 (*Standards and Procedures for referencing Project Elevations Grades to Nationwide Vertical Datums*)

3-6. Orthometric Elevations. Orthometric elevations are those corresponding to the earth's irregular geoidal surface, as illustrated in Figure 3-4. 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. 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 North American Vertical Datum of 1988 (NAVD88); however, other vertical datums may be used in some projects (e.g., the International Great Lakes Datum of 1955 (IGLD55) and the revised International Great Lakes Datum of 1985 (IGLD85). 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. Tidal reference datums (e.g., MLLW) vary geographically over short distances and must be accurately related to NAVD88 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 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.

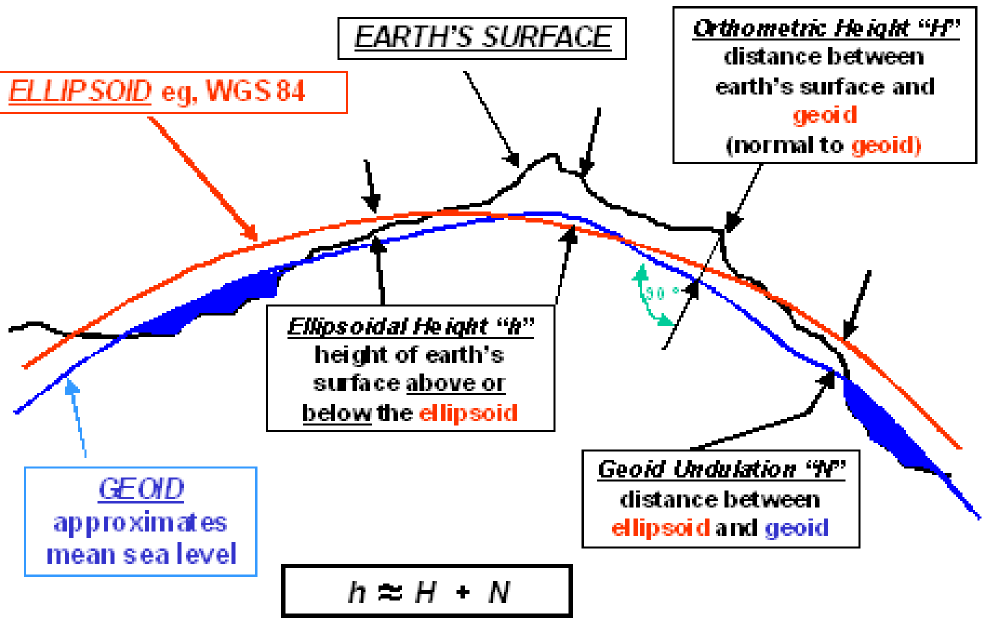

Figure 3-4. Ellipsoid, geoid, and earth's surface definitions and relationships

3-7. Ellipsoidal Heights. In the US, final positions from DGPS are established with respect to NAD83. Since NAD83 is based on the GRS80 ellipsoid, ellipsoid heights obtained from GPS surveying using NAD83 control are based on the GRS80 ellipsoid. These heights are referred to as NAD83 GPS ellipsoidal heights. Unlike the WGS84 ellipsoid, the GRS80 ellipsoid is not exactly geocentric, which can create problems (i.e. large errors) when converting NAD83 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 WGS84 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. 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 NSRS 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. Additional information on the use of GPS for making project connections to the National Spatial Reference System may be found in EM 1110-2-6056 (Standards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums)

3-8. Orthometric Height and WGS84 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 an ellipsoidal height and an orthometric height relative to the geoid can be obtained from the following equation, as depicted graphically in Figure 3-4.

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

where

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

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

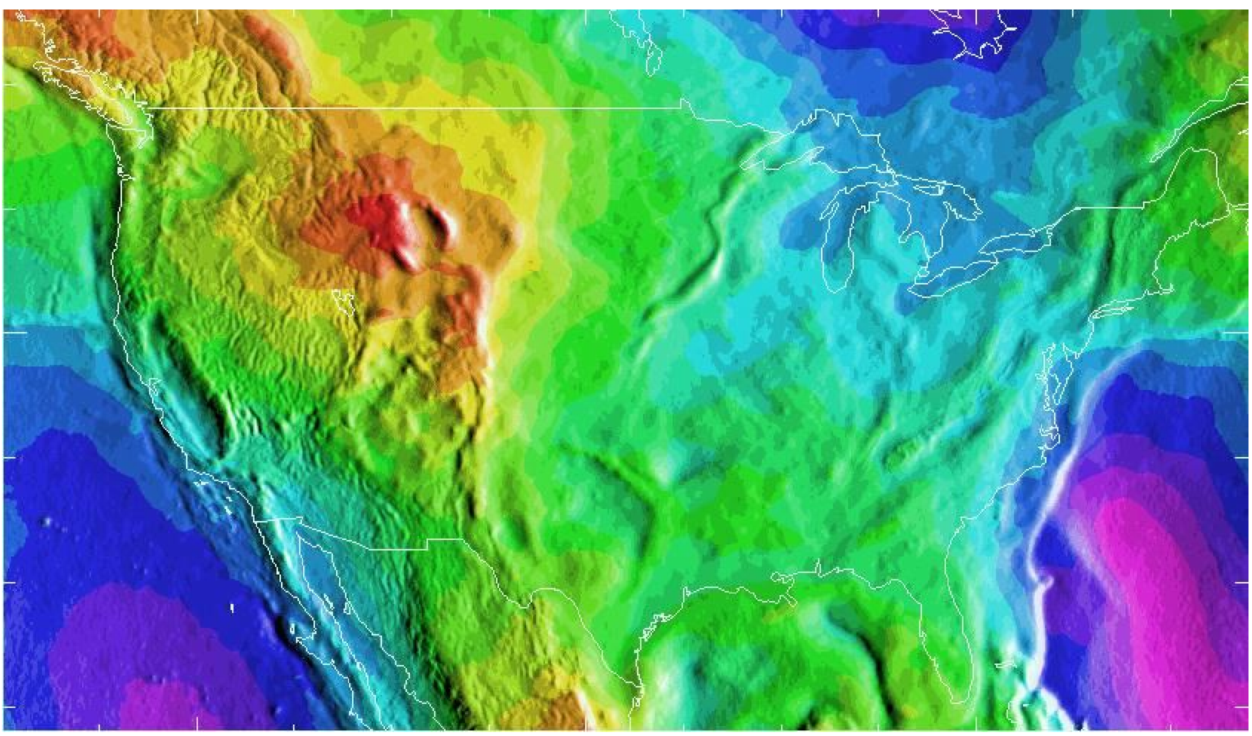

Figure 3-5. Geoid undulation model of North America--depicts geoid undulation "*N*" relative to the WGS84 ellipsoid

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 3-5. 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. Geoid models are not compatible with the superseded NGVD29.

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 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.

d. Gravimetric versus Hybrid Geoid Models. The gravimetric geoid models (e.g., USGG2009) all are based on a GRS80 ellipsoid shell in the ITRF00 reference frame. These models collectively define the same geopotential surface (geoid) determined from the underlying reference global earth gravity model (EGM2008). Hence, comparisons of these heights between different regions provide consistent values. GEOID09 is a hybrid geoid model that can transform between a NAD83 ellipsoid height and the relevant vertical datum for each region. For CONUS and Alaska, a hybrid geoid model can transform from a NAD83 ellipsoid height to a NAVD88 orthometric height. For Puerto Rico, the model can yield a PRVD02 elevation. Each region has its own vertical datum with the exception of Hawaii. Both Geoid03 and Geoid09 determine the NAVD88 surface. The differences arise from the underlying gravimetric geoids (USGG2003 and USGG2009, respectively) and GPSBM data sets available at the time of generation. USGG2003 was based on the EGM96 reference model, while USGG2009 was based on EGM08. EGM08 used the GRACE gravity field product, so it is much more reliable at longer wavelengths. USGG2009 also had better and more consistent terrain models, which affects the shortest wavelengths of the gravity field. The expectation is that USGG2009 is significantly more accurate than USGG2003. The gravimetric geoid determines most of the actual features in the hybrid geoids. GPS-derived ellipsoid heights on leveled bench marks (GPSBM's) give the separation between NAD83 and NAVD88 at discrete points. These differences can be used to develop a conversion surface that effectively warps the gravimetric geoid surface to fit the GPSBM points. The resulting hybrid geoid then fits at the discrete GPSBM points, while still honoring the shorter wavelength features determined by the gravimetric geoid model in between the GPSBM's. Hence, the GPSBM's are also fairly important. GPSBM's continue to change though - their values are not static. There are regions where the ground changes constantly, but most "changes" in height at the BM locations are due to a different realization of the heights. An adjustment is performed using different parameters, and a new value is determined for a height at a given location. The actual position of the height didn't change, just our understanding of where it is located. Since 2003, NGS performed the National Readjustment of 2007. This significantly affected the ellipsoid heights of data all around the country. Since the GPSBM's are determined from the difference between the ellipsoidal (NAD83) and orthometric (NAVD88) heights, a change in one height type changes the control value and thus affects the hybrid geoid. GEOID03 was fit to the data available in 2003. If you are still using data from 2003, the GEOID03 will fit better. However, NGS has changed the heights in the database for many of these points. Hence, GEOID03 will no longer fit the current values to (in some cases) better than the dm-level. GEOID09 was developed using current values. Hence, GEOID09 should be used now if you want to get results consistent with those showing on the NGS datasheets (our current best estimate of actual heights). Both models do yield estimates of NAVD88. However, GEOID09 fits better based on our current understanding of the true position of the coordinates. This will change, too. As NGS develops better coordinates, the values in the database and expressed on the datasheets will change. Hence, GEOID09 will eventually become obsolete and need to be replaced with an update that reflects the future database. (excerpted from http://www.ngs.noaa.gov/GEOID/USGG2009/faq_2009.shtml). It should be noted that where the ellipsoid height values at CORS sites remain essentially unchanged, the implementation of Geoid09 may itself introduce differences in OPUS-derived orthometric height values based on the same observation data set and selected CORS control. The graph below depicts the differences in NAVD88 orthometric heights derived in OPUS solutions based on geoid model selection (Geoid03:05 or Geoid09) in south Louisiana.

3-10. North American Vertical Datum of 1988 (NAVD88). The NAVD88 datum is the product of a vertical adjustment of leveled height difference measurements made across North America. This reference system supersedes the NGVD29 vertical reference framework. NAVD88 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. NAVD88 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. NAVD88 is the most compatible vertical reference frame available to relate GPS ellipsoidal heights to orthometric heights. Note also that NGVD29 is no longer supported by NGS; thus, USACE commands should be transitioning all older project vertical control to NAVD88. The differences in orthometric elevations between the superseded NGVD29 and NAVD88 references are significant--upwards of 1.5 meters in places, as depicted in Figure 3-7. 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.

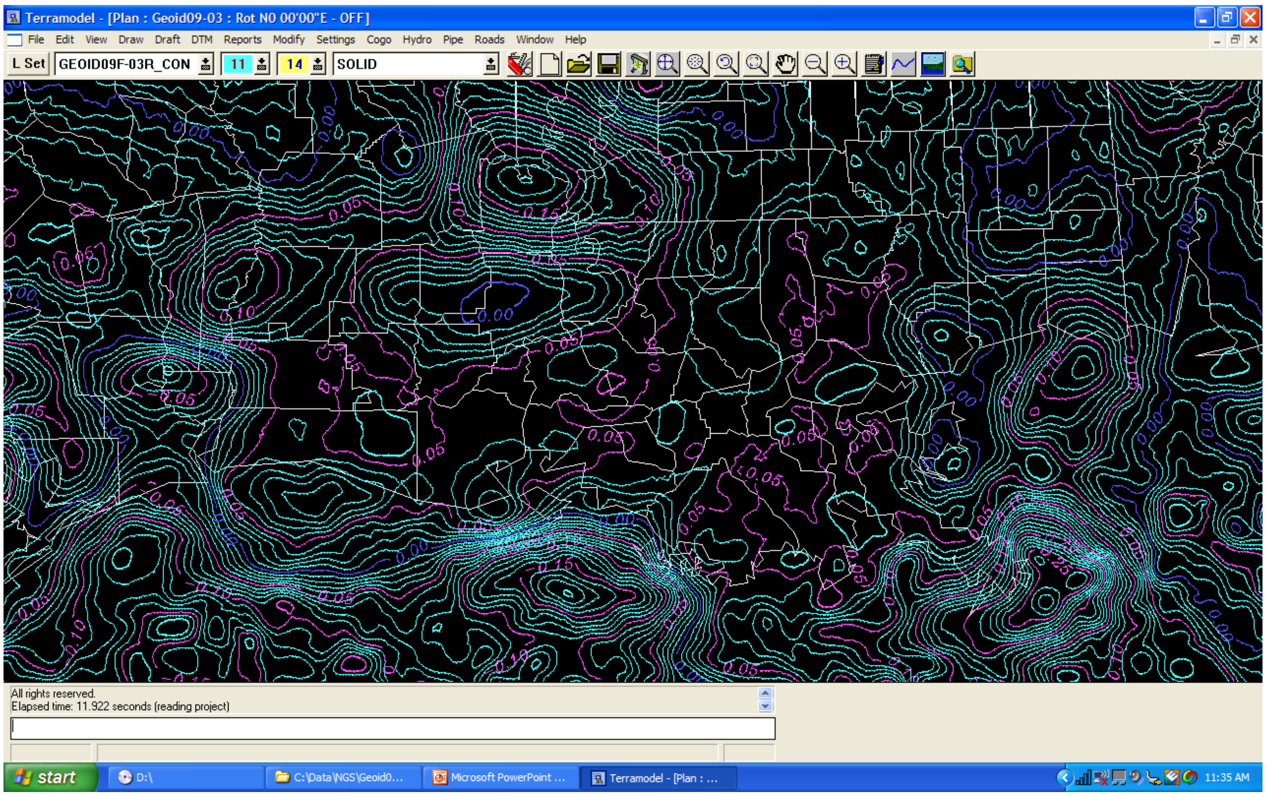

Figure 3-6. Geoid09 minus Goeoid03:05 in South Louisiana (units are meters)

3-11. <u>Using GPS to Densify Orthometric Elevations</u>. 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 accurately determine the ellipsoidal height differences between two GPS stations. Typically, GPS surveys output vertical positions in geodetic coordinates defined with respect to the GRS80 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 GRS80 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. Additional information on the use of GPS for making project connections to the National Spatial Reference System may be found in EM 1110-2-6056 (Standards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums).

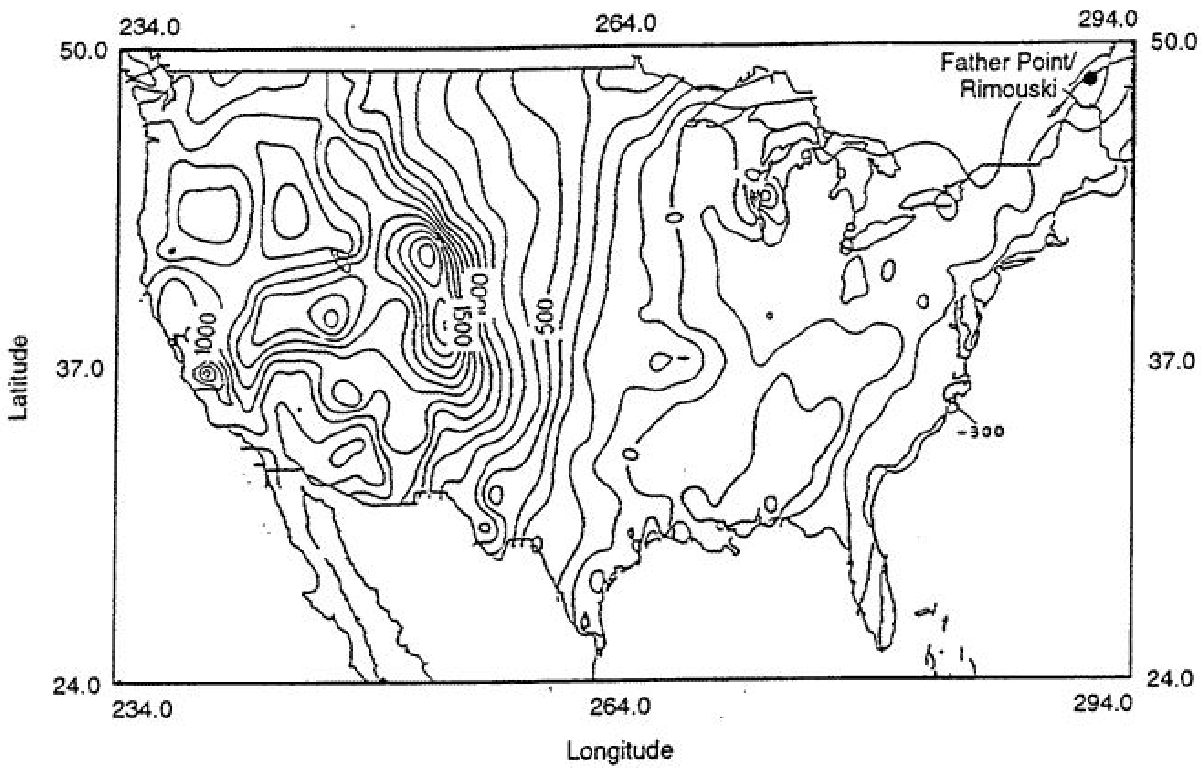

Figure 3-7. NGVD29-NAVD88 elevation differences in mm

3-12. <u>GPS Vertical Site Calibration</u>. 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 systems, 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 coordinates of the points. Points tied to the NSRS 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 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 WGS84 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 NSRS 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 is needed. 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.

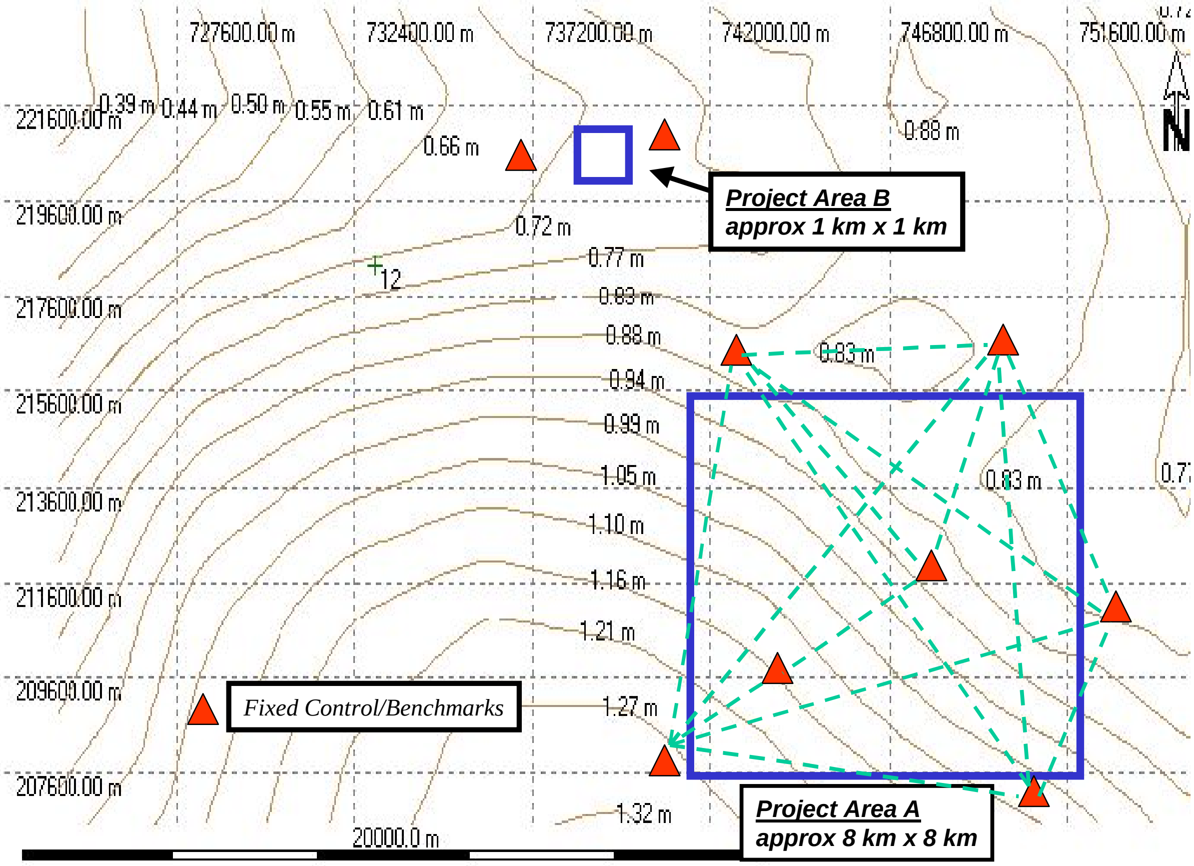

Figure 3-8. Plot of geoid undulation contours over a local survey area (Leica)

c. Figure 3-9 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.

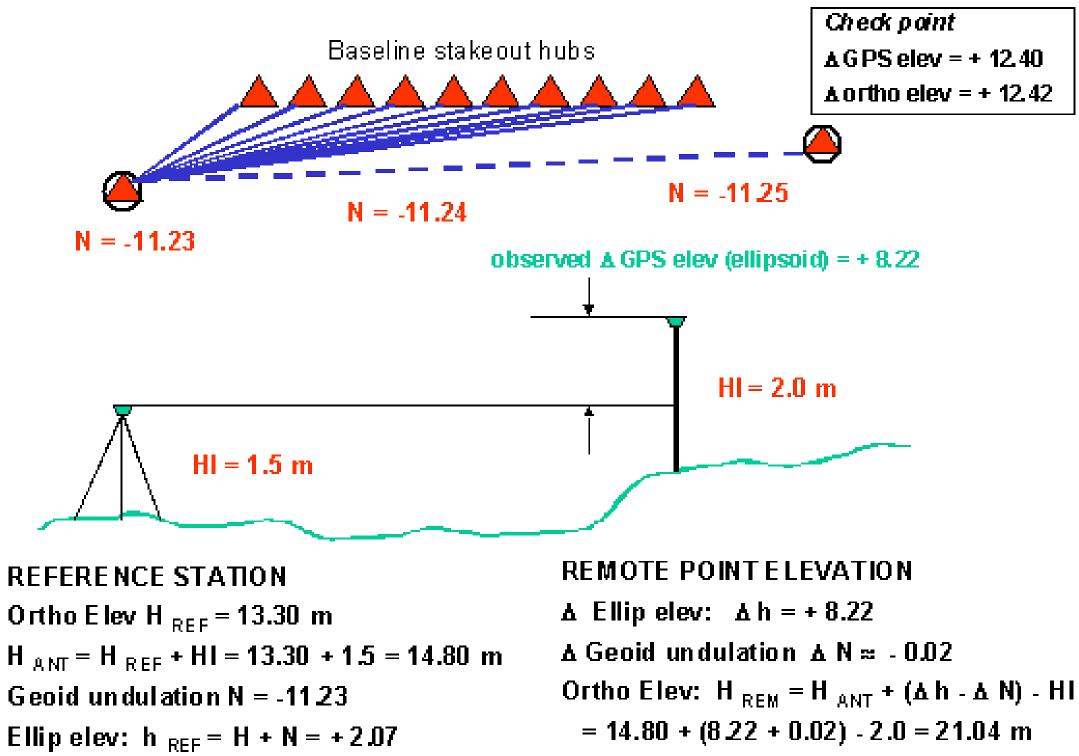

Figure 3-9. Geoid elevation corrections for localized surveys

In the Figure 3-10, geoidal undulation and its importance in computing GPS derived orthometric heights may be seen graphically. In the vertical control extension survey from NSRS station E 299, BA01 SM01, and G 365, ignoring geoid undulation would result in 0.5 feet of vertical error in final orthometric heights at new established stations POND and DAVIS.

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

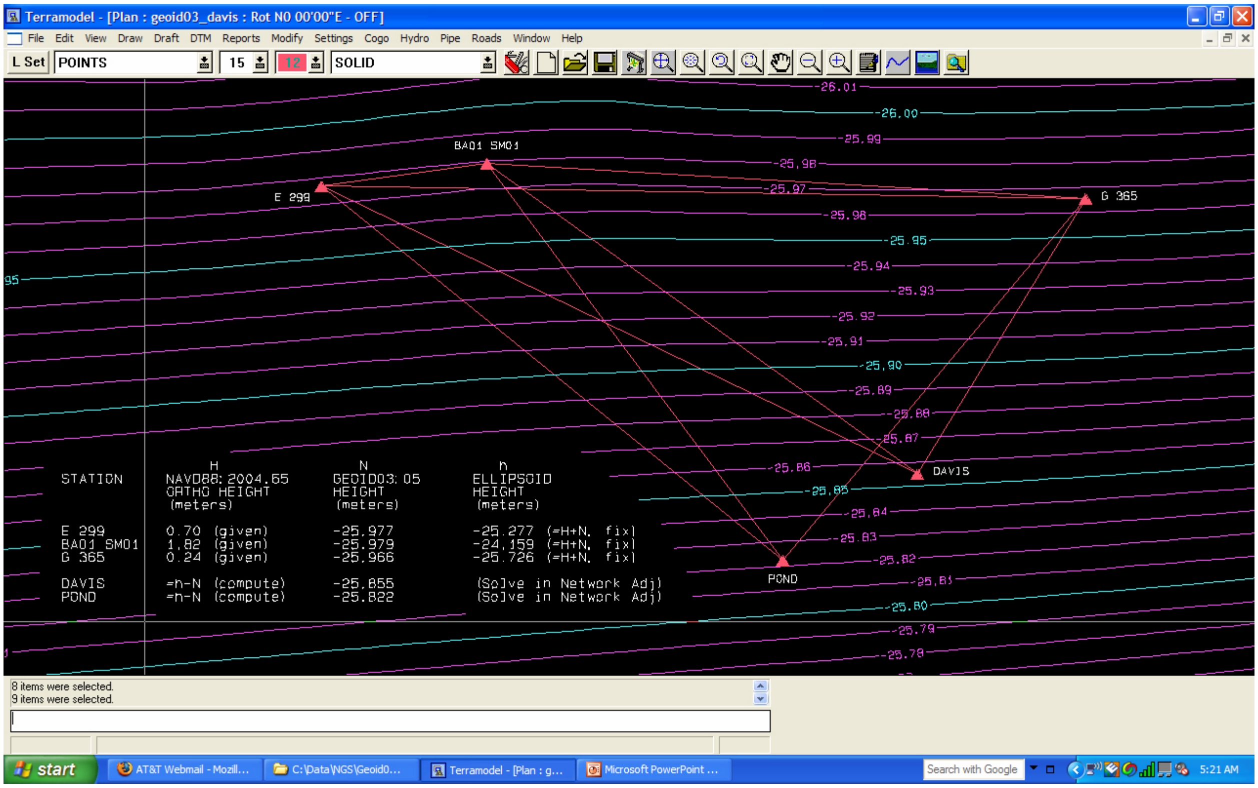

Figure 3-10. Geoid03:05 contours and impact on orthometric height determination

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.