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SUR-111 EXAM PREVIEW

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Exam Preview:

1. Construction plans, maps, facility plans, and CADD/GIS databases are created by a variety of terrestrial, satellite, acoustic, or aerial mapping techniques that acquire planimetric, topographic, hydrographic, or feature attribute data.
    a. True
    b. False
2. Performance-oriented (i.e. outcome based) specifications are recommended in procuring surveying and mapping services. ____ set forth the end results to be achieved (final map format, data content, and/or accuracy standard) and not the means, or technical procedures, used to achieve those results.
    a. Performance specifications
    b. Industry standards
    c. Industry specifications
    d. Industry performance
3. The discipline of surveying consists of locating ____ on the surface of the earth. The positions of ____ are defined by coordinate values that are referenced to a predefined mathematical surface. In geodetic surveying, this mathematical surface is called a datum, and the position of a point with respect to the datum is defined by its coordinates.
    a. Datum points
    b. Points of interest
    c. Geo points
    d. Coordinate points

4. Map accuracies are defined by the positional accuracy of a particular graphical or spatial feature depicted. A map accuracy standard classifies a map as statistically meeting a certain level of accuracy. For most engineering projects, the desired accuracy is stated in the specifications, usually based on the final development scale of the map--both the horizontal "target" scale and vertical relief (specified contour interval or digital elevation model)
    a. True
    b. False
5. NSSDA stands for National Standard for Spatial Datum Accuracy.
    a. True
    b. False
6. Most State and local governments prescribe survey and map accuracy standards. State surveyor licensing boards may prescribe "minimum technical standards" for various real property surveys. State transportation departments may have additional standards unique to their design and construction requirements.
    a. True
    b. False
7. The WGS 84 ellipsoid is used to reference GPS satellite observations and is used to reduce observations onto the NAD 83 system.
    a. True
    b. False
8. A vertical ___ is the surface to which elevations or depths are referred to or referenced. There are many vertical datums used within CONUS.
    a. Datum
    b. Reference point
    c. Nav line
    d. Coordinate system
9. In practice the shape of the geoid surface is estimated globally as a function of horizontal coordinates referenced to a common geocentric position.
    a. True
    b. False
10. "Control" refers to data used in geodesy and cartography to determine the positions and elevations of points on the earth's surface or on a cartographic representation of that surface. A collective term for a system of marks or objects on the earth or on a map or a photograph whose positions or elevation are determined.
    a. True
    b. False

Chapter 4
Accuracy Standards for Engineering, Construction, and Facility Management Control and Topographic Surveys

4-1. Purpose

This chapter sets forth accuracy standards and other related criteria that are recommended for use in large-scale site plan topographic surveys for engineering and construction purposes. These standards relate to surveys performed to locate, align, and stake out construction for civil and military projects, e.g., buildings, utilities, roadways, runways, flood control and navigation projects, training ranges, etc. In many cases, these engineering surveys are performed to provide the base horizontal and vertical control used for area mapping, GIS development, preliminary planning studies, detailed site plan drawings for construction plans, construction measurement and payment, preparing as-built drawings, installation master planning mapping, future maintenance and repair activities, and other AM/FM products. Most engineering surveying standards currently used are based on local practice, or may be contained in State minimum technical standards. The standards given in this chapter conform to the criteria prescribed in EM 1110-1-2909 (*Geospatial Data and Systems*) and the FGDC *Geospatial Positioning Accuracy Standard.* See Appendix A for a list of these FGDC standards.

4-2. General Surveying and Mapping Specifications

Construction plans, maps, facility plans, and CADD/GIS databases are created by a variety of terrestrial, satellite, acoustic, or aerial mapping techniques that acquire planimetric, topographic, hydrographic, or feature attribute data. Specifications for obtaining these data should be "performance-based" and not overly prescriptive or process oriented. They should be derived from the functional project requirements and use recognized industry accuracy standards where available.

a. Industry standards. Maximum use should be made of industry standards and consensus standards established by private voluntary standards bodies--in lieu of Government-developed standards. Therefore, industry-developed accuracy standards should be given preference over Government standards. A number of professional associations have published surveying and mapping accuracy standards, such as the American Society for Photogrammetry and Remote Sensing (ASPRS), the American Society of Civil Engineers (ASCE), the American Congress on Surveying and Mapping (ACSM), and the American Land Title Association (ALTA). When industry standards are non-existent, inappropriate, or do not meet a project's functional requirement, FGDC, DOD, DA, or USACE standards may be specified as criteria sources. Minimum technical standards established by state boards of registration, especially on projects requiring licensed surveyors, should be followed when legally applicable. Local surveying and mapping standards should not be developed where consensus industry standards or DOD/DA standards exist.

b. Performance specifications. Performance-oriented (i.e. outcome based) specifications are recommended in procuring surveying and mapping services. Performance specifications set forth the end results to be achieved (final map format, data content, and/or accuracy standard) and not the means, or technical procedures, used to achieve those results. Performance-oriented specifications typically provide the most flexibility and use of state-of-the-art instrumentation and techniques. Performance specifications should succinctly define only the basic mapping requirements that will be used to verify conformance with the specified criteria, e.g., mapping limits, feature location and attribute requirements, scale, contour interval, map format, sheet layout, and final data transmittal, archiving or storage requirements, required accuracy criteria standards for topographic and planimetric features that are to be depicted, and quality assurance procedures. Performance-oriented specifications should be free from unnecessary equipment, personnel, instrumentation, procedural, or material limitations; except as needed to establish comparative cost estimates for negotiated services.

c. Prescriptive (procedural) specifications. Use of prescriptive specifications should be kept to a minimum, and called for only on highly specialized or critical projects where only one prescribed technical method is appropriate or practical to perform the work. Prescriptive specifications typically require specific field instrumentation, equipment, personnel, office technical production procedures, or rigid project phasing with on-going design or construction. Prescriptive specifications may, depending on the expertise of the writer, reduce flexibility, efficiency, and risk, and can adversely impact project costs if antiquated methods or instrumentation are required. Prescriptive specifications also tend to shift most liability to the Government. Occasionally, prescriptive specifications may be applicable to Corps projects involving specialized work not routinely performed by private surveying and mapping firms, e.g., mapping tactical operation sites, mapping hazardous, toxic, and radioactive waste (HTRW) clean-up sites, military/tactical surveying, or structural deformation monitoring of locks, dams, and other flood control structures.

d. Quality control and quality assurance. Quality control (QC) of contracted surveying and mapping work should generally be performed by the contractor. Therefore, USACE quality assurance (QA) and testing functions should be focused on whether the contractor meets the required performance specification (e.g., accuracy standard), and not the intermediate surveying, mapping, and compilation steps performed by the contractor. The contractor's internal QC will normally include independent tests that may be periodically reviewed by the Government. Government-performed (or monitored) field testing of map accuracies is an optional QA requirement, and should be performed when technically and economically justified, as determined by the ultimate project function.

e. Metrication. Surveying and mapping performed for design and construction should be recorded and plotted in the units prescribed for the project by the requesting Command or project sponsor. During transition to the metric system, inch-pound (IP) units or soft conversions may be required for some geospatial data.

f. Spatial coordinate reference systems. Where practical, feasible, or applicable, civil and military projects should be adequately referenced to nationwide or worldwide coordinate systems directly derived from, or indirectly connected to, GPS satellite observations. In addition, navigation and flood control projects in tidal areas should be vertically referenced to the latest datum epoch established by the Department of Commerce--see Appendix B (*Requirements and Procedures for Referencing Coastal Navigation Projects to Mean Lower Low Water (MLLW) Datum*) for detailed requirements and procedures.

4-3. Accuracy Standards for Engineering and Construction Surveying

a. Accuracy standards. Engineering and construction surveys are normally specified and classified based on the horizontal (linear) point closure ratio or a vertical elevation difference closure standard. This type of performance criteria is most commonly specified in Federal agency, state, and local surveying standards, and should be followed and specified by USACE commands. These standards are applicable to most types of engineering and construction survey equipment and practices (e.g., total station traverses, differential GPS, differential spirit leveling). These accuracy standards are summarized in the following tables.

Table 4-1
Minimum Closure Accuracy Standards for Engineering and Construction Surveys

USACE Classification	Closure Standard	
Engr & Const Control	Distance (Ratio)	Angle (Secs)
First-Order	1:100,000	$2\cdot\sqrt{N}$ [1]
Second Order, Class I	1:50,000	$3\cdot\sqrt{N}$
Second Order, Class II	1:20,000	$5\cdot\sqrt{N}$
Third Order, Class I	1:10,000	$10\cdot\sqrt{N}$
Third Order, Class II	1: 5,000	$20\cdot\sqrt{N}$
Engineering Construction (Fourth-Order)	1: 2,500	$60\cdot\sqrt{N}$

[1] N = Number of angle stations

Table 4-2
Minimum Elevation Closure Accuracy Standards for Engineering and Construction Surveys

USACE Classification	Elevation Closure Standard (ft)[1]	(mm)
First-Order, Class I	$0.013\cdot\sqrt{M}$	$3\cdot\sqrt{K}$
First-Order, Class II	$0.017\cdot\sqrt{M}$	$4\cdot\sqrt{K}$
Second Order, Class I	$0.025\cdot\sqrt{M}$	$6\cdot\sqrt{K}$
Second Order, Class II	$0.035\cdot\sqrt{M}$	$8\cdot\sqrt{K}$
Third Order	$0.050\cdot\sqrt{M}$	$12\cdot\sqrt{K}$
Construction Layout	$0.100\cdot\sqrt{M}$	$24\cdot\sqrt{K}$

[1] $\sqrt{M}$ or $\sqrt{K}$ = square root of distance in Miles or Kilometers

b. Survey closure standards. Survey closure standards listed in Tables 4-1 and 4-2 should be used as a basis for classifying, standardizing, and evaluating survey work. The point and angular closures (i.e. traverse misclosures) relate to the relative accuracy derived from a particular survey. This relative accuracy (or, more correctly, precision) is estimated based on internal closure checks of a traverse survey run through the local project, map, land tract, or construction site. Relative survey accuracy estimates are always expressed as ratios of the traverse/loop closure to the total length of the survey (e.g., 1:10,000).

(1) Horizontal closure standard. The horizontal point closure ratio is determined by dividing the linear distance misclosure of the survey into the overall circuit length of a traverse, loop, or network line/circuit. When independent directions or angles are observed, as on a conventional traverse or closed loop survey, these angular misclosures should be distributed (balanced) before assessing positional misclosure. In cases where differential GPS vectors are measured in three-dimensional geocentric coordinates, then the horizontal component of position misclosure is assessed relative to Table 4-1.

(2) Vertical control standards. The vertical accuracy of a survey is determined by the elevation misclosure within a level section or level loop. For conventional differential or trigonometric leveling, section or loop misclosures (in millimeters or feet) should not exceed the limits shown in Table 4-2, where the line or circuit length is measured in the applicable units. Fourth-Order accuracies are intended for construction layout grading work.

c. Construction survey accuracy standards. Construction survey procedural and accuracy specifications should follow recognized industry and local practices. General procedural guidance is contained in a number of standard commercial texts--e.g., Kavanagh 1997. Accuracy standards for construction surveys will vary with the type of construction, and may range from a minimum of 1:2,500 up to 1:20,000. A 1:2,500 "4th-Order Construction" classification is intended to cover temporary control used for alignment, grading, and measurement of various types of construction, and some local site plan topographic mapping or photo mapping control work. Lower accuracies (1:2,500-1:5,000) are acceptable for earthwork, dredging, embankment, beach fill, and levee alignment stakeout and grading, and some site plan, curb and gutter, utility building foundation, sidewalk, and small roadway stakeout. Moderate accuracies (1:5,000) are used in most pipeline, sewer, culvert, catch basin, and manhole stakeouts, and for general residential building foundation and footing construction, and highway pavement. Somewhat higher accuracies (1:10,000-1:20,000) are used for aligning longer bridge spans, tunnels, and large commercial structures. For extensive bridge or tunnel projects, 1:50,000 or even 1:100,000 relative accuracy alignment work may be required. Grade elevations are usually observed to the nearest 0.01 ft for most construction work, although 0.1 ft accuracy is sufficient for riprap placement, earthwork grading, and small diameter pipe placement. Construction control points are usually marked by semi permanent or temporary monuments (e.g., plastic hubs, P-K nails, iron pipes, wooden grade stakes). Construction control is usually set from existing boundary, horizontal, and vertical control points.

d. Geospatial positioning accuracy standards. Many control surveys are now being efficiently and accurately performed using radial (spur) techniques--e.g., single line vectors from electronic total stations or kinematic differential GPS to monumented control points, topographic feature points, property corners, etc. Since these surveys may not always result in loop closures (i.e. closed traverse) alternative specifications for these techniques must be allowed. This is usually done by specifying a radial positional accuracy requirement. The required positional accuracy may be estimated based on the accuracy of the fixed reference point, instrument, and techniques used. Ratio closure standards in Tables 4-1 and 4-2 may slowly decline as more use is made of nation-wide augmented differential GPS positioning and electronic total station survey methods.

(1) GPS satellite positioning technology allows development of map features to varying levels of accuracy, depending on the type of equipment and procedures employed. Government and commercial augmented GPS systems allows direct, real-time positioning of static AM/FM type features and dynamic platforms (survey vessels, aircraft, etc.). Site plan drawings, photogrammetric control, and related GIS features can be directly constructed from GPS or differential GPS observations, at accuracies ranging from 1 cm to 20 meters (95%).

(2) Accuracy classifications of maps and related GIS data developed by GPS methods can be estimated based on the GPS positioning technique employed. Permanent GPS reference stations (Continuously Operating Reference Stations or CORS) can provide centimeter-level point positioning accuracies over wide ranges; thus providing direct map/feature point positioning without need for preliminary control surveys.

e. Higher-order surveys. Requirements for relative line accuracies exceeding 1:50,000 are rare for most facility engineering, construction, or mapping applications. Surveys requiring accuracies of First-Order (1:100,000) or better (e.g., A- or B-Order) should be performed using FGDC geodetic standards and specifications. These surveys must be adjusted and/or evaluated by the National Geodetic Survey (NGS) if official certification relative to the national network is required.

f. Instrumentation and field observing criteria. In accordance with the policy to use performance-based standards, rigid prescriptive requirements for survey equipment, instruments, or operating procedures are discouraged. Survey alignment, orientation, and observing criteria should rarely be rigidly specified; however, general guidance regarding limits on numbers of traverse stations, minimum traverse course lengths, auxiliary azimuth connections, etc., may be provided for information. For some highly specialized work, such as dam monitoring surveys, technical specifications may prescribe that a general type of instrument system be employed, along with any unique operating, calibration, or recordation requirements. Appendix A contains a number of technical references that may be used.

g. Connections to existing control. Surveys should normally be connected to existing local control or project control monuments/benchmarks. These existing points may be those of any Federal (including Corps project control), State, local, or private agency. Ties to local Corps or installation project control and boundary monuments are absolutely essential and critical to design, construction, and real estate. In order to minimize scale or orientation errors, at least two existing monuments should be connected. It is recommended that Corps surveys be connected with one or more stations on the National Spatial Reference System (NSRS), when practicable and feasible. Connections with local project control that have previously been connected to the NSRS are normally adequate in most cases. Connections with the NSRS shall be subordinate to the requirements for connections with local/project control. Details on these NSRS connections are given in Chapter 6.

h. Survey computations, adjustments, and quality control/assurance. Survey computations, adjustments, and quality control should be performed by the organization responsible for the actual field survey. Contract compliance assessment of a survey should be based on the prescribed point closure standards of internal loops, not on closures with external networks of unknown accuracy. In cases where internal loops are not observed, then assessment must be based on external closures. Specifications should not require closure accuracy standards in excess of those required for the project, regardless of the accuracy capabilities of the survey equipment. Least-squares adjustment methods should be optional for Second-Order or lower-order survey work. Details on network adjustments are covered in EM 1110-1-1003 (*NAVSTAR GPS Surveying*). Professional contractors should not be restricted to rigid computational methods, software, or recording forms. Use of commercial software adjustment packages is strongly recommended.

i. Data recording and archiving. Field survey data may be recorded and submitted either manually or electronically. Manual recordation should follow standard industry practice, using field book formats outlined in various technical manuals.

4-4. Accuracy Standards for Maps and Related Geospatial Products

Map accuracies are defined by the positional accuracy of a particular graphical or spatial feature depicted. A map accuracy standard classifies a map as statistically meeting a certain level of accuracy. For most engineering projects, the desired accuracy is stated in the specifications, usually based on the final development scale of the map--both the horizontal "target" scale and vertical relief (specified contour interval or digital elevation model). Often, however, in developing engineering plans, spatial databases may be developed from a variety of existing source data products, each with differing accuracies--e.g., mixing 1 inch = 60 ft topo plans with 1 inch = 400 ft reconnaissance topo mapping. Defining an "accuracy standard" for such a mixed database is difficult and requires retention (attribution) of the source of each data feature in the base. In such cases the developer must estimate the accuracy of the mapped features.

a. ASPRS Standard. For site mapping of new engineering or planning projects, there are a number of industry and Federal mapping standards that may be referenced in contract specifications. The recommended standard for facility engineering is the ASPRS "Accuracy Standards for Large Scale Maps" (ASPRS 1989). This standard, like most other mapping standards, defines map accuracy by comparing the mapped location of selected well-defined points to their "true" location, as determined by a more accurate, independent field survey. Alternately, when no independent check is feasible or practicable, a map's accuracy may be estimated based on the accuracy of the technique used to locate mapped features--e.g., photogrammetry, GPS, total station, plane table. The ASPRS standard has application to different types of mapping, ranging from wide-area, small-scale, GIS mapping to large-scale construction site plans. It is applicable to all types of horizontal and vertical geospatial mapping derived from conventional topographic surveying or photogrammetric surveys. This standard may be specified for detailed construction site plans that are developed using conventional ground topographic surveying techniques (electronic total stations, plane tables, kinematic GPS). The ASPRS standard is especially applicable to site plan development work involving mapping scales larger than 1:20,000 (1 inch = 1,667 ft); it therefore applies to the more typical engineering map scales in the 1:240 (1 inch = 20 ft) to 1:4,800 (1 inch = 400 ft) range. Its primary advantage over other standards is that it contains more definitive statistical map testing criteria, which, from a contract administration standpoint, is desirable. Using the guidance in Tables 4-3 and 4-4 below, specifications for site plans need only indicate the ASPRS map class, target scale, and contour interval.

b. Horizontal (planimetric) accuracy criteria. The ASPRS planimetric standard compares the root mean square error (RMSE) of the average of the squared discrepancies, or differences in coordinate values between the map and an independent topographic ground survey of higher accuracy (i.e. a check survey). The "limiting RMSE" is defined in terms of meters (feet) at the ground scale rather than in millimeters (inches) at the target map scale. This results in a linear relationship between RMSE and target map scale--as map scale decreases, the RMSE increases linearly. The RMSE is the cumulative result of all errors including those introduced by the processes of ground control surveys, map compilation, and final extraction of ground dimensions from the target map. The limiting RMSE shown in Table 4-3 is the maximum permissible RMSE established by the ASPRS standard. These ASPRS limits of accuracy apply to well-defined map test points only--and only at the specified map scale.

c. Vertical (topographic) accuracy criteria. Vertical accuracy has traditionally been, and currently still is, defined relative to the required contour interval for a map. In cases where digital elevation models (DEM) or digital terrain models (DTM) are being generated, an equivalent contour interval can be specified, based on the required digital point/spot elevation accuracy. The contours themselves may be later generated from a DEM using computer software routines. The ASPRS vertical standard also uses the RMSE statistic, but only for well-defined features between contours containing interpretative elevations, or spot elevation points. The limiting RMSE for Class 1 contours is one-third of the contour interval. Testing for vertical map compliance is also performed by independent, equal, or higher accuracy ground survey methods, such as differential leveling. Table 4-4 summarizes the limiting vertical RMSE for well-defined points, as checked by independent surveys at the full (ground) scale of the map.

Table 4-3a. ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Meters) for Well-Defined Points

Target Map Scale	ASPRS Limiting RMSE in X or Y (Meters)		
Ratio m/m	Class 1	Class 2	Class 3
1:50	0.0125	0.025	0.038
1:100	0.025	0.05	0.075
1:200	0.050	0.10	0.15
1:500	0.125	0.25	0.375
1:1,000	0.25	0.50	0.75
1:2,000	0.50	1.00	1.5
1:2,500	0.63	1.25	1.9
1:4,000	1.0	2.0	3.0
1:5,000	1.25	2.5	3.75
1:8,000	2.0	4.0	6.0
1:10,000	2.5	5.0	7.5
1:16,000	4.0	8.0	12.0
1:20,000	5.0	10.0	15.0
1:25,000	6.25	12.5	18.75
1:50,000	12.5	25.0	37.5
1:100,000	25.0	50.0	75.0
1:250,000	62.5	125.0	187.5

Table 4-3b. ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Feet) for Well-Defined Points

Target Map Scale		ASPRS Limiting RMSE in X or Y (Feet)		
1"= x ft	Ratio ft/ft	Class 1	Class 2	Class 3
5	1:60	0.05	0.10	0.15
10	1:120	0.10	0.20	0.30
20	1:240	0.2	0.4	0.6
30	1:360	0.3	0.6	0.9
40	1:480	0.4	0.8	1.2
50	1:600	0.5	1.0	1.5
60	1:720	0.6	1.2	1.8
100	1:1,200	1.0	2.0	3.0
200	1:2,400	2.0	4.0	6.0
400	1:4,800	4.0	8.0	12.0
500	1:6,000	5.0	10.0	15.0
800	1:9,600	8.0	16.0	24.0
1,000	1:12,000	10.0	20.0	30.0
1,667	1:20,000	16.7	33.3	50.0

Table 4-4a. ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Meters)

	ASPRS Limiting RMSE in Meters					
Target Contour Interval	Topographic Feature Points			Spot or Digital Terrain Model Elevation Points		
Meters	Class 1	Class 2	Class 3	Class 1	Class 2	Class 3
0.10	0.03	0.07	0.10	0.02	0.03	0.05
0.20	0.07	0.13	0.2	0.03	0.07	0.10
0.25	0.08	0.17	0.25	0.04	0.08	0.12
0.5	0.17	0.33	0.50	0.08	0.16	0.25
1	0.33	0.66	1.0	0.17	0.33	0.5
2	0.67	1.33	2.0	0.33	0.67	1.0
4	1.33	2.67	4.0	0.67	1.33	2.0
5	1.67	3.33	5.0	0.83	1.67	2.5
10	3.33	6.67	10.0	1.67	3.33	5.0

Table 4-4b. ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Feet)

	ASPRS Limiting RMSE in Feet					
Target Contour Interval	Topographic Feature Points			Spot or Digital Terrain Model Elevation Points		
ft	Class 1	Class 2	Class 3	Class 1	Class 2	Class 3
0.5	0.17	0.33	0.50	0.08	0.16	0.2
1	0.33	0.66	1.0	0.17	0.33	0.5
2	0.67	1.33	2.0	0.33	0.67	1.0
4	1.33	2.67	4.0	0.67	1.33	2.0
5	1.67	3.33	5.0	0.83	1.67	2.5

d. Map accuracy quality assurance testing and certification. Independent map testing is a quality assurance function that is performed independent of normal quality control during the mapping process. Specifications and/or contract provisions should indicate the requirement (or option) to perform independent map testing. Independent map testing is rarely performed for engineering and construction surveys. If performed, map testing should be completed within a fixed time period after delivery, and if performed by contract, after proper notification to the contractor. In accordance with the ASPRS standard, the horizontal and vertical accuracy of a map is checked by comparing measured coordinates or elevations from the map (at its intended target scale) with spatial values determined by a check survey of higher accuracy. The check survey should be at least twice (preferably three times) as accurate as the map feature tolerance given in the ASPRS tables, and a minimum of 20 points tested. Maps and related geospatial databases found to comply with a particular ASPRS standard should have a statement indicating that standard. The compliance statement should refer to the data of lowest accuracy depicted on the map, or, in some instances, to specific data layers or levels. The statement should clearly indicate the target map scale at which the map or feature layer was developed. When independent testing is not performed, the compliance statement should clearly indicate that the procedural mapping specifications were designed and performed to meet a certain ASPRS map classification, but that a rigid compliance test was not performed. Published maps and geospatial databases whose errors exceed those given in a standard should indicate in their legends or metadata files that the map is not controlled and that dimensions are not to scale. This accuracy statement requirement is especially applicable to GIS databases that may be compiled from a variety of sources containing known or unknown accuracy reliability.

e. National Standard for Spatial Data Accuracy (NSSDA). The traditional small-scale "United States National Map Accuracy Standard" (Bureau of the Budget 1947) has been revised by the FGDC as the NSSDA ("Geospatial Positioning Accuracy Standards, PART 3: National Standard for Spatial Data Accuracy"). This latest version of the NSSDA indicates it is directly based on the ASPRS standard; however, the ASPRS coordinate-based standard is converted to a 95% radial error statistic and the vertical standard is likewise converted from a one-sigma (68%) to 95% standard. The NSSDA defines positional accuracy of spatial data, in both digital and graphic form, as derived from sources such as aerial photographs, satellite imagery, or other maps. Its purpose is to facilitate the identification and application of spatial data by implementing a well-defined statistic (i.e. the 95% confidence level) and testing methodology. As in the ASPRS standard, accuracy is assessed by comparing the positions of well-defined data points with positions determined by higher accuracy methods, such as ground surveys. Unlike the above ASPRS tables, the draft NSSDA standard does not define pass-fail criteria--data and map producers must determine what accuracy exists for their data. Users of that data determine what constitutes acceptable accuracies for their applications. Unlike the ASPRS standard that uses the RMSE statistic in the X, Y, and Z planes, the NSSDA defines horizontal spatial accuracy by circular error of a data set's horizontal (X & Y) coordinates at the 95% confidence level. Vertical spatial data is defined by linear error of a data set's vertical (Z) coordinates at the 95% confidence level. ASPRS lineal horizontal accuracies in X and Y can be converted to NSSDA radial accuracy by multiplying the limiting RMSE values by 2.447, that is:

$$\text{Radial Accuracy}_{NSSDA} = 2.447 \cdot RMSE_{ASPRS\text{-- } X \text{ or } Y} \qquad \text{(Eq 4-1)}$$

ASPRS 1-sigma (68%) vertical accuracies can be converted to NSSDA 95% lineal accuracy by multiplying the limiting RMSE values by 1.96, or:

$$\text{Vertical Accuracy}_{NSSDA} = 1.96 \cdot RMSE_{ASPRS \text{ -- } Z} \qquad \text{(Eq 4-2)}$$

In time, it is expected that the NSSDA will be the recognized standard for specifying the accuracy of all mapping and spatial data products, and the ASPRS standard will be modified to 95% confidence level specifications.

f. Other mapping standards. When work is performed for DOD tactical elements or other Federal agencies or overseas, mapping standards other than ASPRS may be required.

4-5. Photogrammetric Mapping Standards and Specifications

Most smaller scale (e.g., less than 1 inch = 100 ft or 1:1,200) engineering topographic mapping and GIS data base development is accomplished by aerial mapping techniques. The ASPRS standards should be used in specifying photogrammetric mapping accuracy requirements. Procedures for developing photogrammetric mapping specifications are contained in EM 1110-1-1000 (*Photogrammetric Mapping*). This manual contains guidance on specifying flight altitudes, determining target scales, and photogrammetric mapping cost estimating techniques. A full contract guide specification is also contained in an appendix to EM 1110-1-1000.

4-6. Cadastral or Real Property Survey Accuracy Standards

a. General. Many State codes, rules, statutes, or general professional practices prescribe minimum technical standards for real property surveys. Corps in-house surveyors or contractors should follow applicable State technical standards for real property surveys involving the determination of the perimeters of a parcel or tract of land by establishing or reestablishing corners, monuments, and boundary lines, for the purpose of describing, locating fixed improvements, or platting or dividing parcels. Although some State standards relate primarily to accuracies of land and boundary surveys, other types of survey work may also be covered in some areas. Refer to ER 405-1-12, (*Real Estate Handbook*), and the "*Manual of Instructions for the Survey of the Public Lands of the United States*" (US Bureau of Land Management 1973) for additional technical guidance on performing cadastral surveys, or surveys of private lands abutting or adjoining Government lands.

b. ALTA/ACSM standards. Real property survey accuracy standards recommended by ALTA/ACSM are contained in "*Minimum Standard Detail Requirements for ALTA/ACSM Land Title Surveys*" (ALTA 1999), a portion of which is excerpted below. (Note that these ALTA standards are periodically updated--the latest version should be obtained from the reference noted in Appendix A-3). This standard was developed to provide a consistent national standard for land title surveys and may be used as a guide in specifying accuracy closure requirements for USACE real property surveys. However, it should be noted that the ALTA/ACSM standard itself not only prescribes closure accuracies for land use classifications but also addresses specific needs particular to land title insurance matters. The standards contain requirements for detailed information and certification pertaining to land title insurance, including information discoverable from the survey and inspection that may not be evidenced by the public records. The standard also contains a table as to optional survey responsibilities and specifications that the title insurer may require. USACE cadastral surveys not involving title insurance should follow State minimum standards, not ALTA/ACSM standards. On land acquisition surveys which may require title insurance, the decision to perform an ALTA/ACSM standard survey, including all optional survey responsibilities and specifications, should come from the project sponsor. Meeting ALTA/ACSM Urban Class accuracy standards is considered impractical for small tracts or parcels less than 1 acre in size.

Accuracy Standards for ALTA-ACSM Land Title Surveys

Introduction

These Accuracy Standards address Positional Uncertainty and Minimum Angle, Distance and Closure Requirements for ALTA-ACSM Land Title Surveys. In order to meet these standards, the Surveyor must assure that the Positional Uncertainties resulting from the survey measurements made on the survey do not exceed the allowable Positional Tolerance. If the size or configuration of the property to be surveyed or the relief, vegetation, or improvements on the property will result in survey measurements for which the Positional Uncertainty will exceed the allowable Positional Tolerance, the surveyor must alternatively apply the within table of "Minimum Angle, Distance and Closure Requirements for Survey Measurements Which Control Land Boundaries for ALTA-ACSM Land Title Surveys" to the measurements made on the survey or employ, in his or her judgment, proper field procedures, instrumentation and adequate survey personnel in order to achieve comparable results.

The lines and corners on any property survey have uncertainty in location which is the result of (1) availability and condition of reference monuments, (2) occupation or possession lines as they may differ from record lines, (3) clarity or ambiguity of the record descriptions or plats of the surveyed tracts and its adjoiners and (4) Positional Uncertainty.

The first three sources of uncertainty must be weighed as evidence in the determination of where, in the professional surveyor's opinion, the boundary lines and corners should be placed. Positional Uncertainty is related to how accurately the surveyor is able to monument or report those positions.

Of these four sources of uncertainty, only Positional Uncertainty is controllable, although due to the inherent error in any measurement, it cannot be eliminated. The first three can be estimated based on evidence; Positional Uncertainty can be estimated using statistical means.

The surveyor should, to the extent necessary to achieve the standards contained herein, compensate or correct for systematic errors, including those associated with instrument calibration. The surveyor shall use appropriate error propagation and other measurement design theory to select the proper instruments, field procedures, geometric layouts and computational procedures to control and adjust random errors in order to achieve the allowable Positional Tolerance or required traverse closure.

If radial survey methods are used to locate or establish points on the survey, the surveyor shall apply appropriate procedures in order to assure that the allowable Positional Tolerance of such points is not exceeded.

Definitions:

"Positional Uncertainty" is the uncertainty in location, due to random errors in measurement, of any physical point on a property survey, based on the 95% confidence level.

"Positional Tolerance" is the maximum acceptable amount of Positional Uncertainty for any physical point on a property survey relative to any other physical point on the survey, including lead-in courses.

Computation of Positional Uncertainty

The Positional Uncertainty of any physical point on a survey, whether the location of that point was established using GPS or conventional surveying methods, may be computed using a minimally constrained, correctly weighted least squares adjustment of the points on the survey.

Positional Tolerances for Classes of Survey

0.07 feet (or 20mm) + 50ppm

Application of Minimum Angle, Distance, and Closure Requirements

The combined precision of a survey can be statistically assured by dictating a combination of survey closure and specified procedures for an ALTA/ACSM Land Title Survey. ACSM, NSPS and ALTA have adopted the following specific procedures in order to assure the combined precision of an ALTA/ACSM Land Title Survey. The statistical base for these specifications is on file at ACSM and available for inspection.

American Congress On Surveying and Mapping
Minimum Angle, Distance and Closure Requirements for Survey Measurements

Which Control Land Boundaries for ALTA/ACSM Land Title Surveys
(Note 1)

Dir. Reading of Instrument (Note 2)	*Instrument Reading Estimated (Note 3)*	*Number of Observations Per Station (Note 4)*	*Spread From Mean of D&R Not To Exceed (Note 5)*	*Angle Closure Where N=No. of Stations Not To Exceed*	*Linear Closure (Note 6)*	*Distance Measurement (Note 7)*	*Minimum Length of Measurements (Notes 8, 9, 10)*
20" <1'> $\boxed{10''}$	*5" <0.1'> N.A.*	*2 D&R*	*5"<0.1'> $\boxed{5''}$*	*$10''\sqrt{N}$*	*1:15,000*	*EDM or Double tape with Steel Tape*	*(8) 81m, (9) 153m, (10) 20m*

Note (1) All requirements of each class must be satisfied in order to qualify for that particular class of survey. The use of a more precise instrument does not change the other requirements, such as number of angles turned, etc.

Note (2) Instrument must have a direct reading of at least the amount specified (not an estimated reading), i.e.: 20" = Micrometer reading theodolite, <1'> = Scale reading theodolite, $\boxed{10''}$ = Electronic reading theodolite.

Note (3) Instrument must have the capability of allowing an estimated reading below the direct reading to the specified reading.

Note (4) D & R means the Direct and Reverse positions of the instrument telescope, i.e., Urban Surveys require that two angles in the direct and two angles in the reverse position to be measured and meaned.

Note (5) Any angle measured that exceeds the specified amount from the mean must be rejected and the set of angles re-measured.

Note (6) Ratio of closure after angles are balanced and closure calculated.

Note (7) All distance measurements must be made with a properly calibrated EDM or Steel tape, applying atmospheric, temperature, sag, tension, slope, scale factor and sea level corrections as necessary.

Note (8) EDM having an error of 5 mm, independent of distance measured (Manufacturer's specifications).

Note (9) EDM having an error of 10 mm, independent of distance measured (Manufacturer's specifications).

Note (10) Calibrated steel tape.

4-7. Hydrographic Surveying Accuracy Standards

Hydrographic surveys are performed for a variety of engineering, construction, and dredging applications in USACE. Accuracy standards, procedural specifications, and related technical guidance are contained in EM 1110-2-1003 (*Hydrographic Surveying*). This manual should be attached to any A-E contract containing hydrographic surveying work, and must be referenced in construction dredging contracts involving in-place measurement and payment. Standards in this manual apply to Corps river and harbor navigation project surveys, such as dredge measurement and payment surveys, channel condition surveys of inland and coastal Federal navigation projects, beach renourishment surveys, and surveys of other types of marine structures. Accuracy standards are given for different project conditions and depths. Standards for nautical charting surveys or deep-water bathymetric charting surveys should conform to applicable DOD, National Ocean Survey (NOS), or US Naval Oceanographic Office (USNAVOCEANO) accuracy and chart symbolization criteria.

4-8. Structural Deformation Survey Standards

Deformation monitoring surveys of Corps structures require high line vector and/or positional accuracies to monitor the relative movement of monoliths, walls, embankments, etc. Deformation monitoring survey accuracy standards vary with the type of construction, structural stability, failure probability and impact, etc. Since many periodic surveys are intended to measure "long-term" (e.g., monthly or yearly changes) deformations relative to a stable network, lesser survey precisions are required than those needed for short-term structural deflection type measurements. Long-term structural movements measured from points external to the structure may be tabulated or plotted in either X-Y-Z or by single vector movement normal to a potential failure plane. Accuracy standards and procedures for structural deformation surveys are contained in EM 1110-2-1009 (*Structural Deformation Surveying*). Horizontal and vertical deformation monitoring survey procedures are performed relative to a control network established for the structure. Ties to the National Spatial Reference System are not necessary other than for general reference, and then need only USACE Third-Order connection.

4-9. Geodetic Control Survey Standards

Geodetic control surveys are usually performed for the purpose of establishing a basic framework of the National Spatial Reference System (NSRS). These geodetic network densification survey functions are clearly distinct from the traditional engineering and construction surveying and mapping standards covered in this chapter. Geodetic control surveys of permanently monumented control points that are incorporated in the NSRS must be performed to far more rigorous standards and specifications than are control surveys used for general engineering, construction, mapping, or cadastral purposes. When a project requires NSRS densification, or such densification is a desirable by-product and is economically justified, USACE Commands should conform to published FGDC survey standards and specifications. This includes related automated data recording, submittal, project review, and adjustment requirements mandated by FGDC and the National Geodetic Survey. Geodetic survey accuracy and procedural specifications published by the FGDC or NGS include:

- "Standards and Specifications for Geodetic Control Networks" (FGCS 1984)

- "Input Formats and Specifications of the National Geodetic Survey Data Base," NOAA, National Geodetic Survey, (NOAA 1994)

- "Geometric Geodetic Accuracy Standards and Specifications for Using GPS Relative Positioning Techniques (Preliminary)" (FGCS 1988)

- "Guidelines for Submitting GPS Relative Positioning Data to the National Geodetic Survey" (NGS 1988)

- "Geospatial Positioning Accuracy Standards--Part 2: Standards for Geodetic Networks" (FGDC 1998b)

- "Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm)" (NOAA 1997)

Copies of these specifications and standards can be downloaded from the NGS website--see Appendix A. These FGCS/NGS standards and specifications should rarely be specified for Corps control surveys in that they prescribe far more demanding criteria than that needed to establish control for most engineering projects. These FGDC/NGS standards can also easily add 50% or more time and cost to a control survey project.

Part 2 of the FGDC *Geospatial Positioning Accuracy Standards* (Standards for Geodetic Networks)--FGDC 1988b-- prescribes a positional accuracy criteria instead of the traditional linear closure (misclosure) criteria. It is expected that this positional accuracy standard will gradually replace the misclosure standards in Tables 4-1 and 4-2. This new standard is excerpted in Table 4-5 below.

Table 4-5. FGDC Part 2 Accuracy Standards for Geodetic Networks Horizontal, Ellipsoid Height, and Orthometric Height

Accuracy Classification	95-Percent Confidence Less Than or Equal to:
1-Millimeter	0.001 meters
2-Millimeter	0.002 "
5-Millimeter	0.005 "
1-Centimeter	0.010 "
2-Centimeter	0.020 "
5-Centimeter	0.050 "
1-Decimeter	0.100 "
2-Decimeter	0.200 "
5-Decimeter	0.500 "
1-Meter	1.000 "
2-Meter	2.000 "
5-Meter	5.000 "
10-Meter	10.000 "

NOTE: The classification standard for geodetic networks is based on accuracy. Accuracies are categorized separately according to horizontal, ellipsoid height, and orthometric height. Note: although the largest entry in the table is 10 meters, the accuracy standards can be expanded to larger numbers if needed.

4-10. State and Local Accuracy Standards

Most State and local governments prescribe survey and map accuracy standards. These are usually similar to those standards given in the previous tables in this chapter. State surveyor licensing boards may prescribe "minimum technical standards" for various real property surveys. State transportation departments may have additional standards unique to their design and construction requirements.

a. General state surveying and mapping standards. Below is an excerpt of surveying accuracy standards taken from the Florida Administrative Code (FAC 2003). These standards for general boundary surveys are representative of minimum technical standards used by many states.

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(1) Survey and Map Accuracy

(a) REGULATIONAL OBJECTIVE: The public must be able to rely on the accuracy of measurements and maps produced by a surveyor and mapper. In meeting this objective, surveyors and mappers must achieve the following minimum standards of accuracy, completeness, and quality.

(b) The accuracy of the survey measurements shall be premised upon the type of survey and the expected use of the survey and map. All measurements must be in accordance with the United States standard, using either feet or meters. Records of these measurements shall be maintained for each survey by either the individual surveyor and mapper or the surveying and mapping business entity. Measurement and computation records must be dated and must contain sufficient data to substantiate the survey map and insure that the accuracy portion of these standards has been met.

(c) Vertical Control: Field-measured control for elevation information shown upon survey maps shall be based on a level loop. Closure in feet must be accurate to a standard of plus or minus .05 ft. times the square root of the distance in miles. All surveys and maps with elevation data shall indicate the datum and a description of the benchmark(s) upon which the survey is based. Minor elevation data may be obtained on an assumed datum provided the base elevation of the datum is obviously different than the established datum.

(d) Vertical Feature Accuracy:

1. If contour lines are shown, then sufficient data must be obtained in order to insure that 90% of ground point elevations taken from contours are within 1/2 of the contour interval, and the remainder are not in error more than the contour interval.

2. For surveys performed by photogrammetric methods, vertical positional accuracy of map elevations, contours, or other forms of terrain models must be stated. The stated accuracy is a plus or minus tolerance that encompasses 90% of elevation differences between survey measured values and ground truth. All such survey maps or reports with elevation data shall have a statement to the effect: "Elevations of well-identified features contained in this survey been measured to an estimated vertical positional accuracy of: ____ (ft) (m)." If different accuracy levels exist for different features, then applicable features and accuracies shall be identified with similar statements.

(e) Horizontal Control: All surveys and maps expressing or displaying features in coordinate position shall indicate the coordinate datum and a description of the control points upon which the survey is based. Minor coordinate data may be obtained on an assumed datum provided the numerical basis of the datum is obviously different than an established datum. The accuracy of field-measured control measurements shall be statistically verified by measurement and calculation of a closed geometric figure. All control measurements shall be made with a transit and steel tape, or devices with equivalent or higher degrees of accuracy. The relative distance accuracy must be better than the following:

- *Commercial/High Risk Linear: 1 foot in 10,000 feet;*
- *Suburban: Linear: 1 foot in 7,500 feet;*
- *Rural: Linear: 1 foot in 5,000 feet;*

*(f) Horizontal Feature Accuracy (for surveys by photogrammetric methods only): A survey and map's horizontal positional accuracy must be stated. The stated accuracy is a plus or minus tolerance that encompasses 90% of coordinate differences between survey measured values and ground truth. All survey maps or reports shall have a statement of the effect: "Well-identified features in this survey and map have been measured to an estimated horizontal positional accuracy of *[____] (ft) (m)." If different accuracy levels exist for different features, then applicable features and accuracies shall be identified with similar statements.*

(g) Map Plotting Accuracy: The horizontal position of physical features surveyed by field methods must be plotted to within 1/20 of an inch at the map scale.

*(h) Intended Display Scale: At the maximum intended display scale, a survey and map's positional accuracy value occupies 1/20" on the display. All maps or reports of surveys produced by photogrammetric methods and delivered with digital coordinate files must contain a statement to the effect of: "This map is intended to be displayed at a scale of 1/ *[______] or smaller."*

(2) Other Provisions that Apply to All Surveys and Maps.

(a) REGULATIONAL OBJECTIVE: In order to avoid misuse of a survey and map, the surveyor and mapper must adequately communicate the survey results to the public through a map, report, or report with an attached map. Any survey map or report must identify the responsible surveyor and mapper and contain standard content. In meeting this objective, surveyors and mappers must meet the following minimum standards of accuracy, completeness, and quality:

(b) Each survey map and report shall state the type of survey it depicts consistent with the types of surveys defined in Rule 61G17-6.002(8)(a)-(k), F.A.C. The purpose of a survey, as set out in Rule 61G17-6.002(8)(a)-(l), F.A.C., dictates the type of survey to be performed and depicted, and a licensee may not avoid the minimum standards required by rule of a particular survey type merely by changing the name of the survey type to conform with what standards or lack of them the licensee chooses to follow.

(c) All survey maps and reports must bear the name, certificate of authorization number, and street and mailing address of the business entity issuing the map and report, along with the name and license number of the surveyor and mapper in responsible charge. The name, license number, and street and mailing address of a surveyor and mapper practicing independent of any business entity must be shown on each survey map and report.

(d) All survey maps must reflect a survey date, which is the date of the field survey or the date of image acquisition for photogrammetric surveys. If the graphics of a map are revised, but the survey date stays the same, the map must list dates for all revisions.

(l) Responsibility Clearly Stated. The responsibility for all mapped features must be clearly depicted on any map or report signed by a Florida licensed surveyor and mapper. In the case that features surveyed by the signing surveyor and mapper have been integrated with features surveyed by others, then the full extent of responsibility shall be clearly depicted on the map or report, and the signing surveyor and mapper shall include in the map or report an assessment of the quality and accuracy of all mapped features delivered.

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b. DOT control survey standards. The following Third-Order survey standards shown in Figures 4-1and 4-2 below are from the CALTRANS *Surveys Manual*. The first standard is for establishing permanent Third-Order horizontal control using a total station. The second is a Third-Order standard for differential leveling--covering different types of levels. Figure 4-3 depicts a CALTRANS accuracy standard for setting primary control around a project site. The classifications and closure standards are identical with those in Tables 4-1 and 4-2. The "G" classification is roughly comparable to USACE "4th Order" classification

Specifications	Traverse/Network Resection Double Tie
Check vertical index error	Daily
Check horizontal collimation	Daily
Measure instrument height and target height	Begin and end of each setup
Use plummet to check position of target and instrument over points	Begin and end of each setup
Measure temperature and pressure and enter ppm correction into total station	First set-up of day
Measure distance to backsight and foresight at each setup	Required
Observe traverse multiple ties to improve least squares adjustment	As Feasible
Close all traverses	Required
Horizontal angle observations, minimum	3D, 3R
Vertical angle observations, minimum	3D, 3R
Angular rejection limit, residual not to exceed	5"
Maximum value for the standard error of the mean	1.2"
Minimum distance measurement to meet horizontal accuracy standard	50 m
Minimum number of distance measurements	3
Distance rejection limit: residual not to exceed	2mm + 2 ppm
Maximum distance measurement to meet vertical accuracy standard	100 m

Figure 4-1. CALTRANS Third-Order horizontal control standards (Total Station)

Operation/Specification	Compensator-Level Three-Wire Observation	Compensator-Level Single-Wire Observation	Electronic/Digital Bar Code Level
Difference in length between fore and back sights, not to exceed per setup	10 m	10 m	10 m
Cumulative difference in length between fore and backsights, not to exceed per loop or section	10 m	10 m	10 m
Maximum sight lengths	90 m	90 m	90 m (See Note 1)
Minimum ground clearance of sight line	0.5 m	0.5 m	0.5 m
Maximum section misclosure	$12\text{ mm} \times \sqrt{D}$ (See Note 2)	$12\text{ mm} \times \sqrt{D}$ (See Note 2)	$8\text{ mm} \times \sqrt{D}$ (See Note 2)
Maximum loop misclosure	$12\text{ mm} \times \sqrt{E}$ (See Note 3)	$12\text{ mm} \times \sqrt{E}$ (See Note 3)	$8\text{ mm} \times \sqrt{E}$ (See Note 3)
Difference between top and bottom interval not to exceed	0.30 of rod unit	N/A	N/A
Collimation (Two-Peg) Test	Daily (See Note 4) (not to exceed 2 mm)	Daily	Daily
Minimum number of readings (Use repeat measure option for each observation)	N/A	N/A	3 (See Note 5)

Figure 4-2. CALTRANS Third-Order differential leveling standards

CALTRANS ORDER (Note 1)	STANDARDS — CLASSICAL — HORIZONTAL (Note 4)	STANDARDS — CLASSICAL — VERTICAL (Note 5)	STANDARDS — POSITIONAL	MONUMENT SPACING AND SURVEY METHODS (Note 2) — MONUMENT SPACING (MINIMUM)	MONUMENT SPACING AND SURVEY METHODS (Note 2) — TYPICAL SURVEY METHOD — HORIZONTAL	MONUMENT SPACING AND SURVEY METHODS (Note 2) — TYPICAL SURVEY METHOD — VERTICAL	APPLICATION – TYPICAL SURVEYS — HORIZONTAL	APPLICATION – TYPICAL SURVEYS — VERTICAL
B (Note 3)	1:1,000,000	Not Applicable	Per NGS Specifications	10 k	GPS: Static	Not Applicable	High Precision Geodetic Network (HPGN)	Not Applicable
First (Note 3)	1:100,000 (Note 10)	$e = 5\sqrt{E}$	Per NGS Specifications	3 k	GPS: Static Fast Static	Bar Code	Basic (Corridor) Control – HPGN-D Project Control – Horizontal (perferred, when feasible)	Rarely used. Crustal Motion Surveys, etc.
Second	1:20,000	$e = 8\sqrt{E}$	(Note 8)	500 m	GPS: Static Fast Static TSSS: Net Traverse	Bar Code 3-Wire TSSS: Trig	Project Control – Horizontal (see First Order also)	Basic (Corridor) Control HPGN and HPGN-D Project Control
Third	1:10,000	$e = 12\sqrt{E}$	(Note 8)	As Required	GPS: Static Fast Static Kinematic RTK (Note 13) TSSS: Net Traverse Resection Double Tie (Note 9)	Bar Code Single Wire TSSS: Trig GPS: Static (Note 7) RTK (Note 11)	Supplemental Control > Engineering > Construction • Interchange • Major Structure Photo. Control – Horizontal Right of Way Surveys Construction Surveys (Note 6) Topographic Surveys (Note 6) Major Structure Points (Staked)	Project Control – Vertical Supplemental Control Photo. Control – Vertical Construction Surveys (Note 6) Topographic Surveys (Note 6) Major Structure Points (Staked)
G (General)	As required, see appropriate survey procedure section in this manual for accuracy standards/tolerances.			Not Applicable	GPS: Fast Static Kinematic RTK TSSS: Radial	GPS: Fast Static Kinematic, RTK (Note 12) TSSS: Trig Single Wire Direct Elevation Rod	Topographic Surveys (Data Points) Supplement Design Data Surveys Construction Surveys (Staked Points) Environmental Surveys GIS Data Surveys Right of Way Flagging	

Notes

1. The standards, specifications, and procedures included in this Manual are based on Federal Geodetic Control Subcommittee (FGCS) standards and specifications. Except where otherwise noted, the FGCS requirements have been modified to meet Caltrans needs.
2. Refer to other Manual sections for detailed procedural specifications for specific survey methods and types of surveys.
3. "B" Order and First Order surveys are performed to FGCS standards and specifications or other requirements approved by National Geodetic Survey.
4. Distance accuracy standard.
5. Closure between established control; e = maximum misclosure in mm, E = distance in km.
6. Survey setup points used for radial stake out.
7. For example a static GPS may be used to establish NAV88 at the project site from a distant NAVD National Spatial Reference System Control.
8. As required by the local survey needs.
9. Instead of including a point as a network point, certain survey points may be positioned by observations from two or more control points (i.e., double tied). If survey points are not included in a network, double ties must be performed to ensure that blunders are eliminated and the positions established are within stated accuracy standard. Double tie procedures should be only used when appropriate; possible examples are photo control points, land net and monumentation points, and major structure stake points.
10. The distance accuracy standard for Basic (Corridor) Control – HPGN-D surveys is 1:500,000.
11. Not to include vertical project control or vertical for major structure points.
12. Not to include pavement elevations.
13. Not to include major structures.

Figure 4-3. CALTRANS accuracy classifications and standards

4-11. CADD/GIS Technology Center Standards

The CADD/GIS Technology Center [for Facilities, Infrastructure, and Environment] is located at the USACE Waterways Experiment Station in Vicksburg, MS. The Center's primary mission is to serve as a multi-service vehicle to set computer-aided design and drafting (CADD) and geographic information system (GIS) standards; coordinate CADD/GIS facilities systems within the Department of Defense (DOD); promote CADD/GIS system integration; support centralized CADD/GIS hardware and software acquisition; and provide assistance for the installation, training, operation, and maintenance of CADD and GIS systems. The intent of the CADD/GIS Technology Center standards development initiatives has been to develop usable CADD, GIS, and facility management (FM) standards that will satisfy the project life-cycle concept for digital data. This concept requires a set of CADD, GIS, and FM standards for initial data collection, analysis, design, construction, and subsequent master planning, facility management, and maintenance. This allows for direct integration from CADD engineering design or as-builts to such GIS analysis tasks as master planning and FM. The Center has issued a number of geospatial standards and related CADD/GIS guidance. Some of these standards include:

- Spatial Data Standard for Facilities, Infrastructure, and Environment (SDSFIE)
- Facility Management Standard for Facilities, Infrastructure, and Environment (FMSFIE)
- A/E/C CADD Standard

These A/E/C CADD standards define symbology, level/layer assignments, drafting templates, sheet layouts, and other criteria required in a CADD environment. The SDSFIE Standards define the attributes and attribute values for geospatial data features. These standards should be specified for in-house or A-E services requiring delivery of CADD, GIS, and other spatial and geospatial data covered by this chapter.

4-12. Mandatory Standards

The accuracy standards in the following tables in this chapter are considered mandatory.

- Table 4-1: Minimum Closure Accuracy Standards for Engineering and Construction Surveys

- Table 4-2: Minimum Elevation Closure Accuracy Standards for Engineering and Construction Surveys

- Table 4-3a: ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Meters) for Well-Defined Points

- Table 4-3b: ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Feet) for Well-Defined Points

- Table 4-4a: ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Meters)

- Table 4-4b: ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Feet)

HQUSACE has directed that geospatial data collected for architectural, engineering, and construction projects shall be compliant with the A/E/C CADD Standard and/or the SDSFIE Standard. This includes data that is collected, developed, or contracted for, and/or otherwise executed by the Corps of Engineers. Topographic survey data falls within this directive.

Chapter 5
Geodetic Reference Datums and Local Coordinate Systems

5-1. Purpose and Background

This chapter provides guidance on geodetic reference datums, coordinate systems, and local horizontal and vertical reference systems that are used to georeference Corps military construction and civil works projects. Use of State Plane Coordinate Systems (SPCS) is covered in detail since these systems are most commonly used to reference topographic surveys of local projects. Transformations between datums and coordinate systems are discussed. Site calibration techniques needed for RTK topographic surveys are covered in Chapter 9.

a. Topographic surveys can be performed on any coordinate system. Most localized total station topographic surveys are initiated on (or referenced to) an arbitrary coordinate grid system, e.g., X=5,000 ft, Y=5,000 ft, Z=100 ft, and often elevation or scale reductions are ignored. Planimetric and topographic data points collected on this arbitrary grid in a data collector are then later translated, rotated, scaled, and/or "best fit" to some established geographical reference system--e.g., the local State Plane Coordinate System (SPCS).

b. The process of converting the observed topographic points on the arbitrary grid system to an established geographical reference system (e.g., SPCS) is termed a “datum transformation.” In order to perform this transformation, a few points (preferably three or more) in the topographic database must be referenced to the external reference system. These “control” points on a topographic survey have been previously established relative to an installation or project’s primary control network. They normally were established using more accurate “geodetic control” survey procedures, such as differential leveling, static or kinematic DGPS observations, or total station traverse.

c. Most USACE topographic surveys require “control surveys” to bring in a geodetic reference network to the local project site where detailed topographic surveys are performed. It is important that the correct geodetic reference network is used, and that it is consistent with the overall installation or project reference system. It may also important that these reference systems conform to regional or nationwide reference systems, such as the National Spatial Reference System (NSRS), North American Datum of 1983 (NAD 83), or the North American Vertical Datum of 1988 (NAVD 88). These various reference datums and systems are discussed in this chapter.

d. Not all topographic surveys require a rigid reference to some local or regional geographic coordinate system, and thus do not need time consuming and expensive preliminary control surveys. Some project feature or on going construction applications may only require a simple local reference--for example, a single monument with assumed or scaled coordinates and an arbitrary reference azimuth may suffice.

e. Other topographic surveys outside Army installations or Corps civil project areas may require rigid references to established property boundaries (corner pins, section corners, road intersections/centerlines, etc.). These ties to legal boundaries and corners will thus establish the reference system by which all topographic survey features are detailed. Regional geodetic or SPCS networks may or may not be required on such surveys, depending on local practice or statute.

f. For additional details on geodetic datums and coordinate systems, refer to EM 1110-1-1003 (*NAVSTAR GPS Surveying*) or consult one of the technical references listed in Appendix A.

SECTION I
Geodetic Reference Systems

5-2. General

The discipline of surveying consists of locating points of interest on the surface of the earth. The positions of points of interest are defined by coordinate values that are referenced to a predefined mathematical surface. In geodetic surveying, this mathematical surface is called a datum, and the position of a point with respect to the datum is defined by its coordinates. The reference surface for a system of control points is specified by its position with respect to the earth and its size and shape. A datum is a coordinate surface used as reference figure for positioning control points. Control points are points with known relative positions tied together in a network. Densification of the network refers to adding more fixed control points to the network. Both horizontal and vertical datums are commonly used in surveying and mapping to reference coordinates of points in a network. Reference systems can be based on the geoid, ellipsoid, or a plane. The physical earth's gravity force can be modeled to create a positioning reference frame that rotates with the earth. The geoid is such a surface (an equipotential surface of the earth's gravity field) that best approximates Mean Sea Level (MSL). The orientation of this surface at a given point on geoid is defined by the plumb line. The plumb line is oriented tangent to the local gravity vector. Surveying instruments can be readily oriented with respect to the gravity field because its physical forces can be sensed with simple mechanical devices. Such a reference surface is developed from an ellipsoid of revolution that best approximates the geoid. An ellipsoid of revolution provides a well-defined mathematical surface to calculate geodetic distances, azimuths, and coordinates.

5-3. Geodetic Coordinates

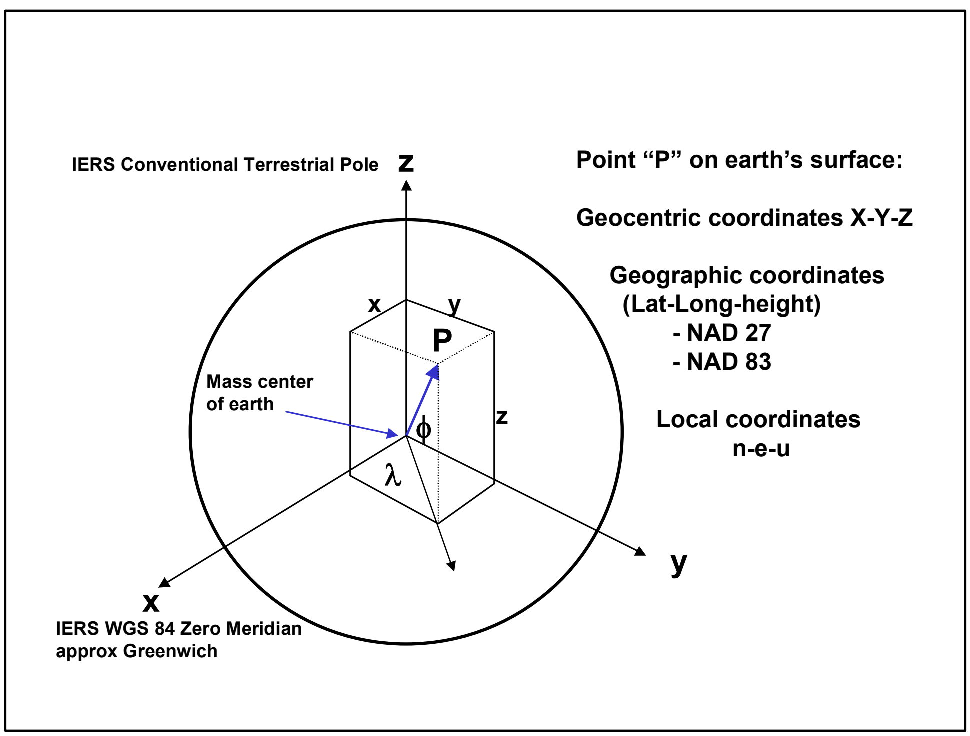

Figure 5-1. Earth-centered earth-fixed coordinate reference frames

A coordinate system is defined by the location of the origin, orientation of its axes, and the parameters (coordinate components) which define the position of a point within the coordinate system. Terrestrial coordinate systems are widely used to define the position of points on the terrain because they are fixed to the earth and rotate with it. The origin of terrestrial systems can be specified as either geocentric (origin at the center of the earth, such as NAD 83) or topocentric (origin at a point on the surface of the earth, such as NAD 27). The orientation of terrestrial coordinate systems is described with respect to its poles, planes, and axes. The primary pole (Z in Figure 5-1 above) is the axis of symmetry of the coordinate system, usually parallel to the rotation axis of the earth, and coincident with the semi-minor axis of the reference ellipsoid. The reference planes that are perpendicular to the primary polar axis are the equator (zero latitude) and the Greenwich meridian plane (zero longitude). Parameters for point positioning within a coordinate system refer to the coordinate components of the system (either Cartesian or curvilinear).

a. Geocentric coordinates. Geocentric coordinates have an origin at the center of the earth, as shown above in Figure 5-1. GPS coordinates are initially observed on this type of reference system. For example, a coordinate on such a system might be displayed on a GPS receiver as:

```
X =    668400.506 m
Y =  -4929214.152 m
Z =   3978967.747 m
```

GPS receivers will transform these geocentric coordinates into a geographic coordinate system described below.

b. Geodetic or Geographic coordinates. Geographic coordinate components consist of:

- latitude (ϕ),
- longitude (λ),
- ellipsoid height (h).

Geodetic latitude, longitude, and ellipsoid height define the position of a point on the surface of the Earth with respect to some “reference ellipsoid.” The most common reference ellipsoid used today is the WGS 84, which will be described in more detail in a later section.

(1) Geodetic latitude (ϕ). The geodetic latitude of a point is the acute angular distance between the equatorial plane and the normal through the point on the ellipsoid measured in the meridian plane (Figure 5-1). Geodetic latitude is positive north of the equator and negative south of the equator.

(2) Geodetic longitude (λ). The geodetic longitude is the angle measured counter-clockwise (east), in the equatorial plane, starting from the prime meridian (Greenwich meridian), to the meridian of the defined point (Figure 5-1). In the continental United States, longitude is commonly reported as a west longitude. To convert easterly to westerly referenced longitudes, the easterly longitude must be subtracted from 360 deg.

East-West Longitude Conversion:

$$\lambda (W) = [\, 360 - \lambda (E) \,] \qquad \text{(Eq 5-1)}$$

For example:

$\lambda (E) = 282^{d}\ 52^{m}\ 36.345^{s}$ E

λ (W) = [360 ^{d} - 282 ^{d} 52 ^{m} 36.345 ^{s} E]
λ (W) = 77 ^{d} 07 ^{m} 23.655 ^{s} W

(3) Ellipsoid Height (h). The ellipsoid height is the linear distance above the reference ellipsoid measured along the ellipsoidal normal to the point in question. The ellipsoid height is positive if the reference ellipsoid is below the topographic surface and negative if the ellipsoid is above the topographic surface.

(4) Geoid Separation (N). The geoid separation (or often termed "geoidal height") is the distance between the reference ellipsoid surface and the geoid surface measured along the ellipsoid normal. The geoid separation is positive if the geoid is above the ellipsoid and negative if the geoid is below the ellipsoid.

(5) Orthometric Height (H). The orthometric height is the vertical distance of a point above or below the geoid.

5-4. Datums

A datum is a coordinate surface used as reference for positioning control points. Both horizontal and vertical datums are commonly used in surveying and mapping to reference coordinates of points in a network.

a. Geodetic datum. Five parameters are required to define an ellipsoid-based datum. The major semi-axis (a) and flattening (f) define the size and shape of the reference ellipsoid; the latitude and longitude of an initial point; and a defined azimuth from the initial point define its orientation with respect to the earth. The NAD 27 and NAD 83 systems are examples of horizontal geodetic datums.

b. Horizontal datum. A horizontal datum is defined by specifying (1) the 2D geometric surface (plane, ellipsoid, sphere) used in coordinate, distance, and directional calculations, (2) the initial reference point (origin), and (3)a defined orientation, azimuth or bearing from the initial point. The “horizontal datum” for most topographic surveys is usually defined relative to the fixed control points (monuments and/or benchmarks) that were used to control the individual shots. These “control points” may, in turn, be referenced to a local installation/compound control network and/or to a regional NSRS CORS station.

c. Project datum. A project datum is defined relative to local control and might not be directly referenced to a geodetic datum. Project datums are usually defined by a system with perpendicular axes, and with arbitrary coordinates for the initial point, and with one (principal) axis oriented toward an assumed north. A chainage-offset system may also be used as a reference, with the PIs (points of intersection) either marked points or referenced to some other coordinate system.

d. Vertical datum. A vertical datum is a reference system used for reporting elevations. The two most common nationwide systems are the National Geodetic Vertical Datum of 1929 (NGVD 29) and the North American Vertical Datum of 1988 (NAVD 88). Vertical elevations used on navigation, flood control, and hydropower projects may also be referenced to a variety of datums, such as:

- Mean Sea Level (MSL)
- Mean Low Water (MLW)
- Mean Lower Low Water (MLLW)
- Mean High Water (MHW)
- International Great Lakes Datum (IGLD)

- Low Water Reference Plane (LWRP)
- Flat Pool Stage
- Local Pool or Reservoir Capacity Reference Point

Mean Sea Level (MSL) based elevations are used for most construction and topographic surveys--in particular those involving flood control or shoreline improvement/protection. It should be noted that MSL elevations are not the same as NGVD 29; and that MSL and NGVD 29 elevations can widely differ from NAVD 88 elevations--as much as 3 ft in western CONUS. MLLW elevations are used in referencing coastal navigation projects. MHW elevations are used in construction projects involving bridges and other crossings over navigable waterways.

e. The National Spatial Reference System (NSRS). The NSRS is that component of the National Spatial Data Infrastructure (NSDI) - [http://www.fgdc.gov/nsdi/nsdi.html] which contains all geodetic control contained in the National Geodetic Survey (NGS) database. This includes: A, B, First, Second and Third-Order horizontal and vertical control, geoid models, precise GPS orbits and Continuously Operating Reference Stations (CORS), and the National Shoreline as observed by NGS as well as data submitted by other Federal, State, and local agencies, academic institutions, and the private sector.

5-5. WGS 84 Reference Ellipsoid

The WGS 84 ellipsoid is used to reference GPS satellite observations and is used to reduce observations onto the NAD 83 system. The origin of the WGS 84 Cartesian system is the earth's center of mass, as shown in Figure 5-1. 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. 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.

Prior to development of WGS 84, there were several reference ellipsoids and interrelated coordinate systems (datums) that were used by the surveying and mapping community. Table 5-1 lists just a few of these reference systems along with their mathematical defining parameters. Note that GRS 80 is the actual reference ellipsoid for NAD 83; however, the difference between GRS-80 and WGS 84 ellipsoids is insignificant. 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 5-1. 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

5-6. Horizontal Datums and Reference Frames

The following paragraphs briefly describe the most common datums used to reference CONUS projects.

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. 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). (Figure 5-2). 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. 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. Positional differences between NAD 83 (1986) and NAD 83 (HARN) can approach 1 meter.

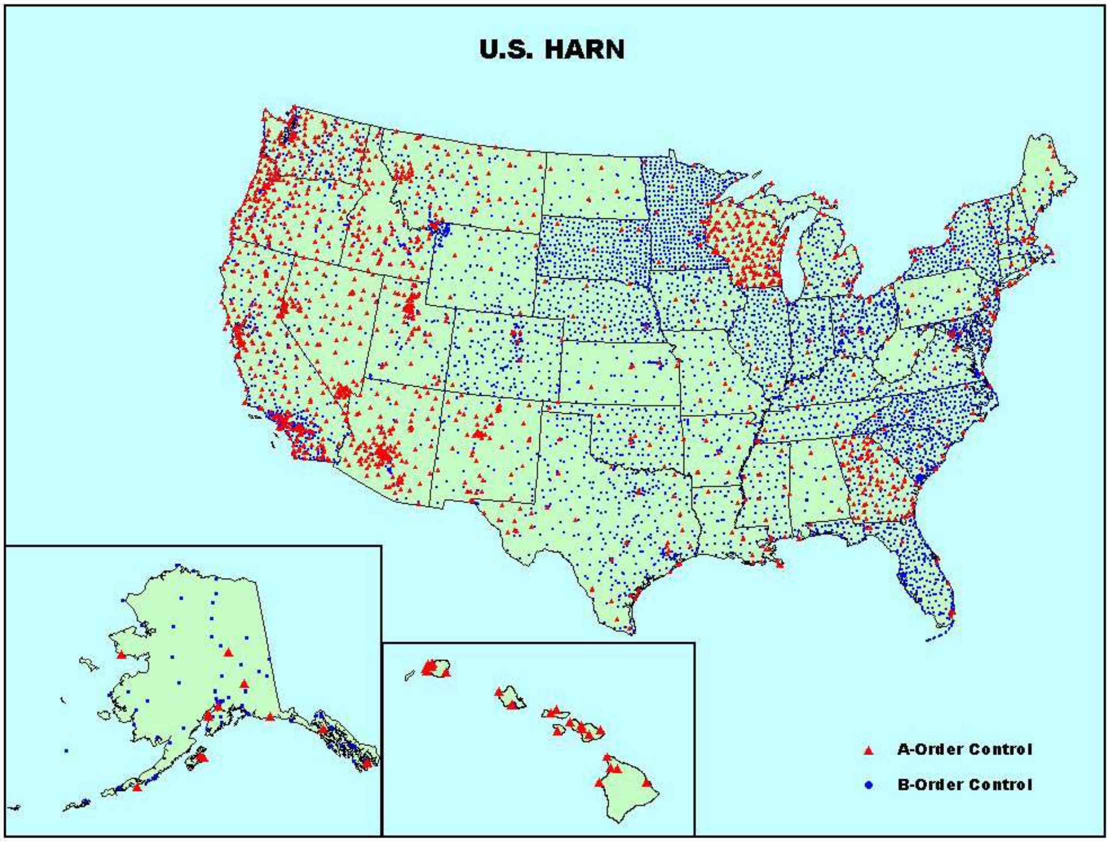

Figure 5-2. High Accuracy Reference Network control points

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 5-3 below. 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, positional difference 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)--see EM 1110-1-1003. CORS are particularly useful when precise control is required in a remote area, from which a topographic survey may be performed. With only 1 to 2 hours of static DGPS observations, reference points can often be established to an ellipsoid accuracy better than ± 0.2 ft in X-Y-Z.

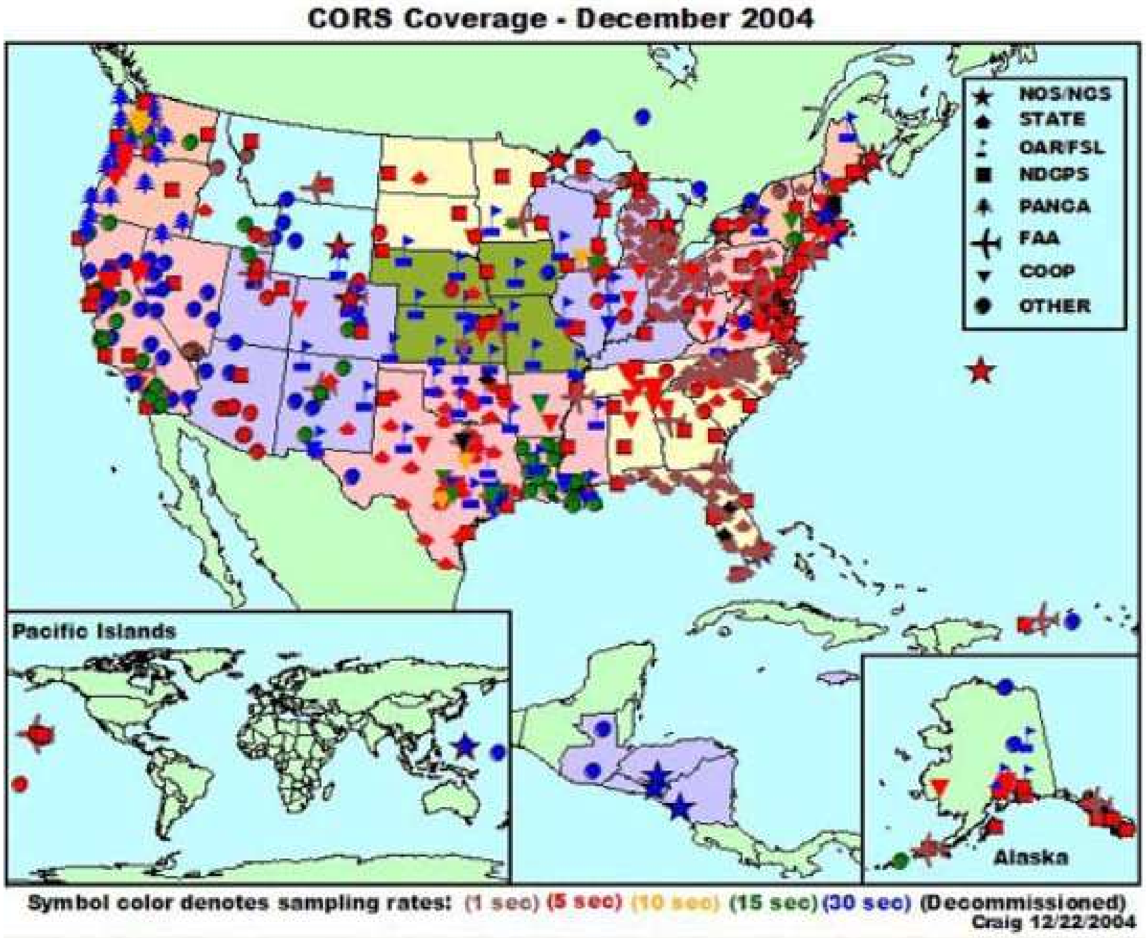

Figure 5-3. Continuously Operating Reference Stations as of 2001 (NGS)

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, and may become significant when geodetic control is established from remote reference stations. These refinements can be used to 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 on control surveys would be rare, and would never be applicable to local facility mapping surveys. Those obtaining coordinates from NGS datasheets must take care not to use ITRF values. The relationship between ITRF, NAD 83, and the geoid is illustrated in Figure 5-4 below.

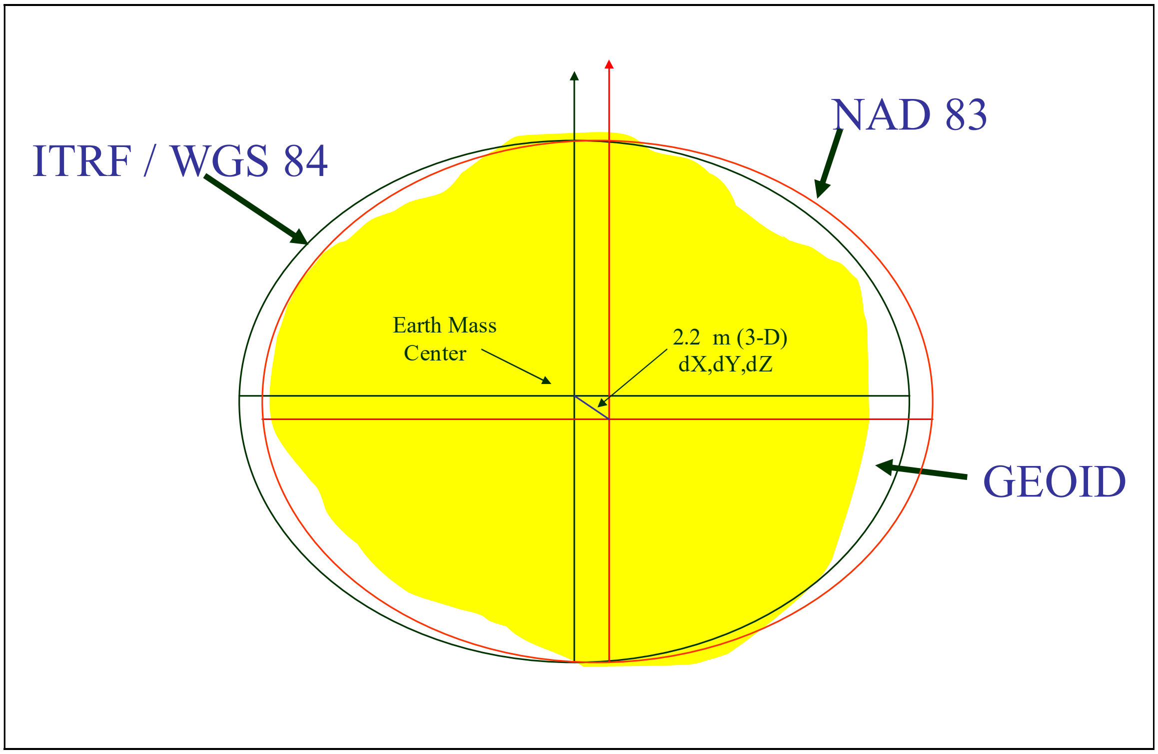

Figure 5-4. Relationship between ITRF, NAD 83, and the geoid

SECTION II
Horizontal Coordinate Systems

5-7. General

Geocentric, geographic, or geodetic coordinates described above are rarely used to reference site plan topographic surveys or maps. Engineering site plan drawings are normally referenced to a local state plane coordinate system (SPCS), or in some cases a metric-based UTM system. In rare cases, they may be referenced to an arbitrary coordinate system relative to some point on the project--a monument, corner, road intersection, etc. In most cases, control surveys performed for setting project control will be computed and adjusted using the SPCS. The following paragraphs describe horizontal coordinate systems commonly used on facility site plan mapping and related control surveys.

5-8. Geographic Coordinates

The use of geographic coordinates as a system of reference is accepted worldwide. It is based on the expression of position by latitude (parallels) and longitude (meridians) in terms of arc (degrees, minutes, and seconds) referred to the equator (north and south) and a prime meridian (east and west). The degree of accuracy of a geographic reference (GEOREF) is influenced by the map scale and the accuracy requirements for plotting and scaling. Examples of GEOREFs are as follows:

40° N 132° E (referenced to degrees of latitude and longitude).
40°21' N 132°14' E (referenced to minutes of latitude and longitude).
40°21'12" N 132°14'18" E (referenced to seconds of latitude and longitude).
40°21'12.4" N 132°14'17.7" E (referenced to tenths of seconds of latitude and longitude).
40°21'12.45" N 132°14'17.73" E (referenced to hundredths of seconds of latitude and longitude).

US military maps and charts include a graticule (parallels and meridians) for plotting and scaling geographic coordinates. Graticule values are shown in the map margin. On maps and charts at scales of 1:250,000 and larger, the graticule may be indicated in the map interior by lines or ticks at prescribed intervals (for example, scale ticks and interval labeling at the corners of 1:50,000 at 1minute [in degrees, minutes, and seconds] and again every 5 minutes).

5-9. State Plane Coordinate Systems

a. General. State Plane Coordinate Systems (SPCS) were developed by the National Geodetic Survey (NGS) to provide plane coordinates over a limited region of the earth's surface. To properly relate geodetic coordinates (ϕ-λ-h) of a point to a 2D plane coordinate representation (Northing, Easting), a conformal mapping projection must be used. Conformal projections have mathematical properties that preserve differentially small shapes and angular relationships to minimize the errors in the transformation from the ellipsoid to the mapping plane. Map projections that are most commonly used for large regions are based on either a conic or a cylindrical mapping surface (Figure 5-5 below). The projection of choice is dependent on the north-south or east-west areal extent of the region. Areas with limited east-west dimensions and indefinite north-south extent use the Transverse Mercator (TM) type projection. Areas with limited north-south dimensions and indefinite east-west extent use the Lambert projection. The SPCS is designed to minimize the spatial distortion at a given point to approximately one part in ten thousand (1:10,000). To satisfy this criteria, the SPCS has been divided into zones that have a maximum width or height of approximately one hundred and fifty eight statute miles (158 miles). Therefore, each state may have several zones or may employ both the Lambert (conic) and Transverse Mercator (cylindrical) projections. The projection state plane coordinates are referenced to a specific geodetic datum (i.e. the datum that the initial geodetic coordinates are referenced to must be known).

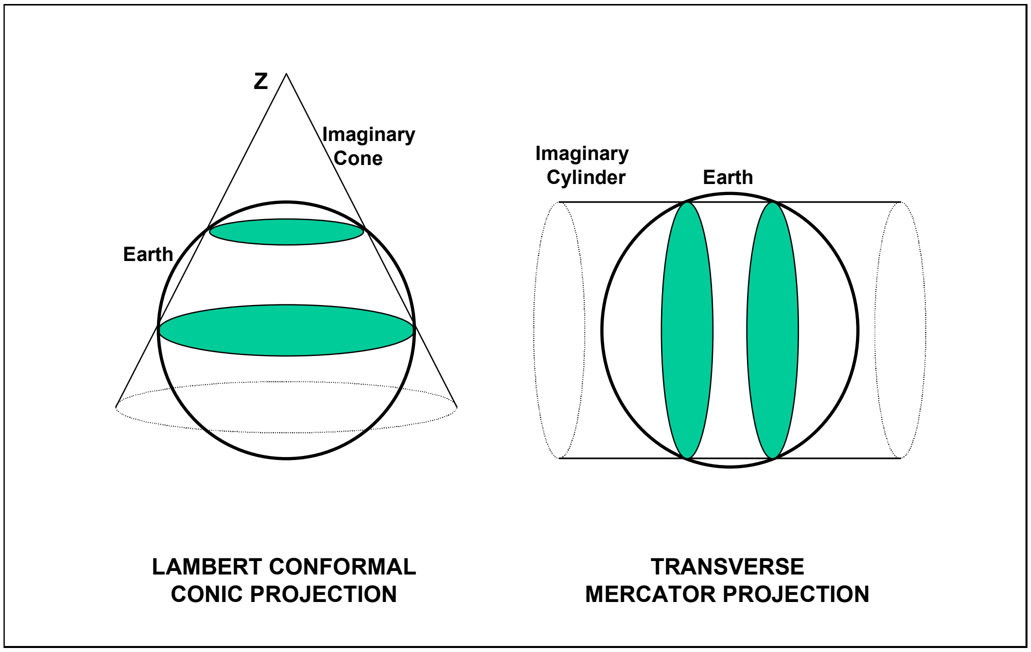

Figure 5-5. Common map projections

b. Transverse Mercator (TM). The Transverse Mercator projection uses a cylindrical surface to cover limited zones on either side of a central reference longitude. Its primary axis is rotated perpendicular to the symmetry axis of the reference ellipsoid. Thus, the TM projection surface intersects the ellipsoid along two lines equidistant from the designated central meridian longitude (Figure 5-6). Distortions in the TM projection increase predominantly in the east-west direction. The scale factor for the Transverse Mercator projection is unity where the cylinder intersects the ellipsoid. The scale factor is less than one between the lines of intersection, and greater than one outside the lines of intersection. The scale factor is the ratio of arc length on the projection to arc length on the ellipsoid. To compute the state plane coordinates of a point, the latitude and longitude of the point and the projection parameters for a particular TM zone or state must be known.

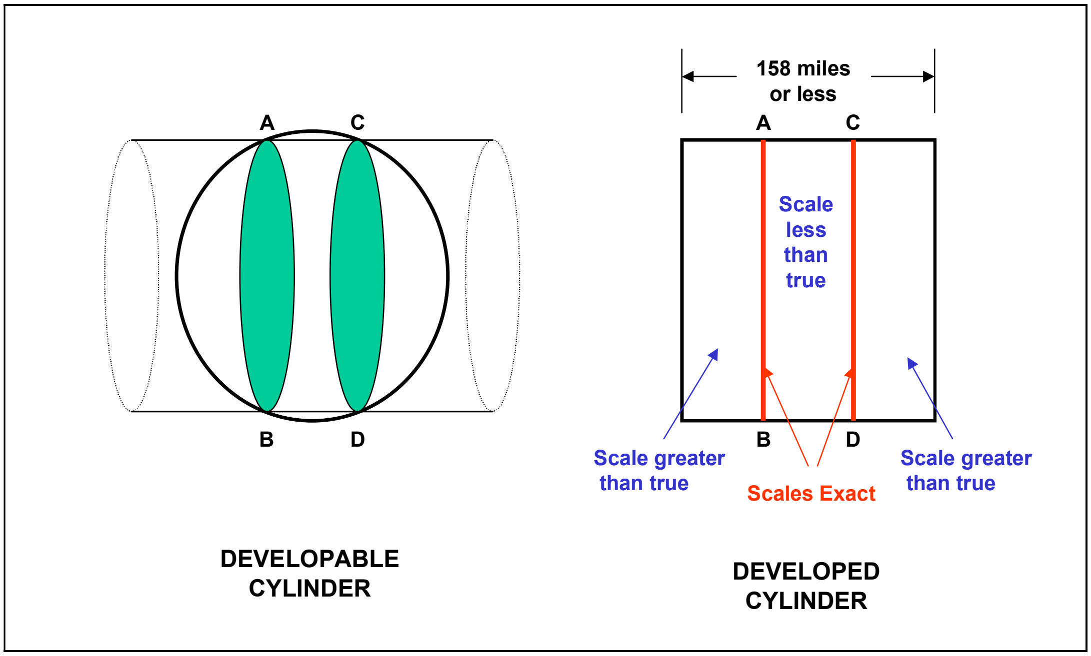

Figure 5-6. Transverse Mercator Projection

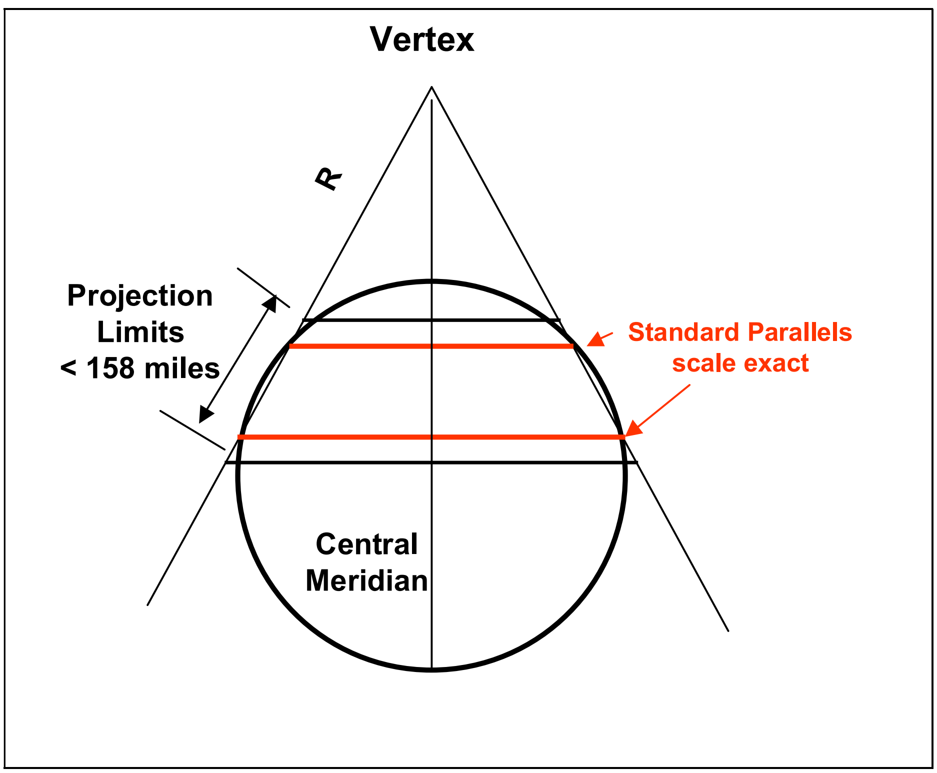

Figure 5-7. Lambert Projection

c. Lambert Conformal Conic (LCC). The Lambert projection uses a conic surface to cover limited zones of latitude adjacent to two parallels of latitude. Its primary axis is coincident with the symmetry axis of the reference ellipsoid. Thus, the LCC projection intersects the ellipsoid along two standard parallels (Figure 5-7). Distortions in the LCC projection increase predominantly in the north-south direction. The scale factor for the Lambert projection is equal to unity at each standard parallel and is less than one inside, and greater than one outside the standard parallels. The scale factor remains constant along the standard parallels.

d. SPCS zones. Figure 5-8 depicts the various SPCS zones in the US. The unique state zone number provides a standard reference when using transformation software developed by NGS and the COE. The state zone number remains constant in both NAD27 and NAD83 coordinate systems.

Figure 5-8. SPCS zones identification numbers for the various states

e. Scale units. State plane coordinates can be expressed in both feet and meters. State plane coordinates defined on the NAD 27 datum are published in feet. State plane coordinates defined on the NAD 83 datum are published in meters; however, state and federal agencies can request the NGS to provide coordinates in feet. If NAD 83 based state plane coordinates are defined in meters and the user intends to convert those values to feet, the proper meter-feet conversion factor must be used. Some states use the International Survey Foot rather than the US Survey Foot in the conversion of feet to meters.

International Survey Foot:

1 International Foot = 0.3048 meter (exact)

US Survey Foot:

1 US Survey Foot = 1200 / 3937 meter (exact)

5-10. Grid Elevations, Scale Factors, and Convergence

In all planer grid systems, the grid projection only approximates the ellipsoid (or roughly the ground), and "ground-grid" corrections must be made for measured distances or angles (directions). Measured ground distances must be corrected for (1) elevation (sea level factor), and (2) ground to grid plane (scale factor). Figure 5-9 below illustrates a reduction of a measured distance (D) down to the ellipsoid distance (S). Not shown is the subsequent reduction from the ellipsoid length to a grid system length. Observed directions (or angles) must also be corrected for grid convergence. Also shown on the figure is the relationship between ellipsoid heights (h), geoid heights (N), and orthometric heights (H).

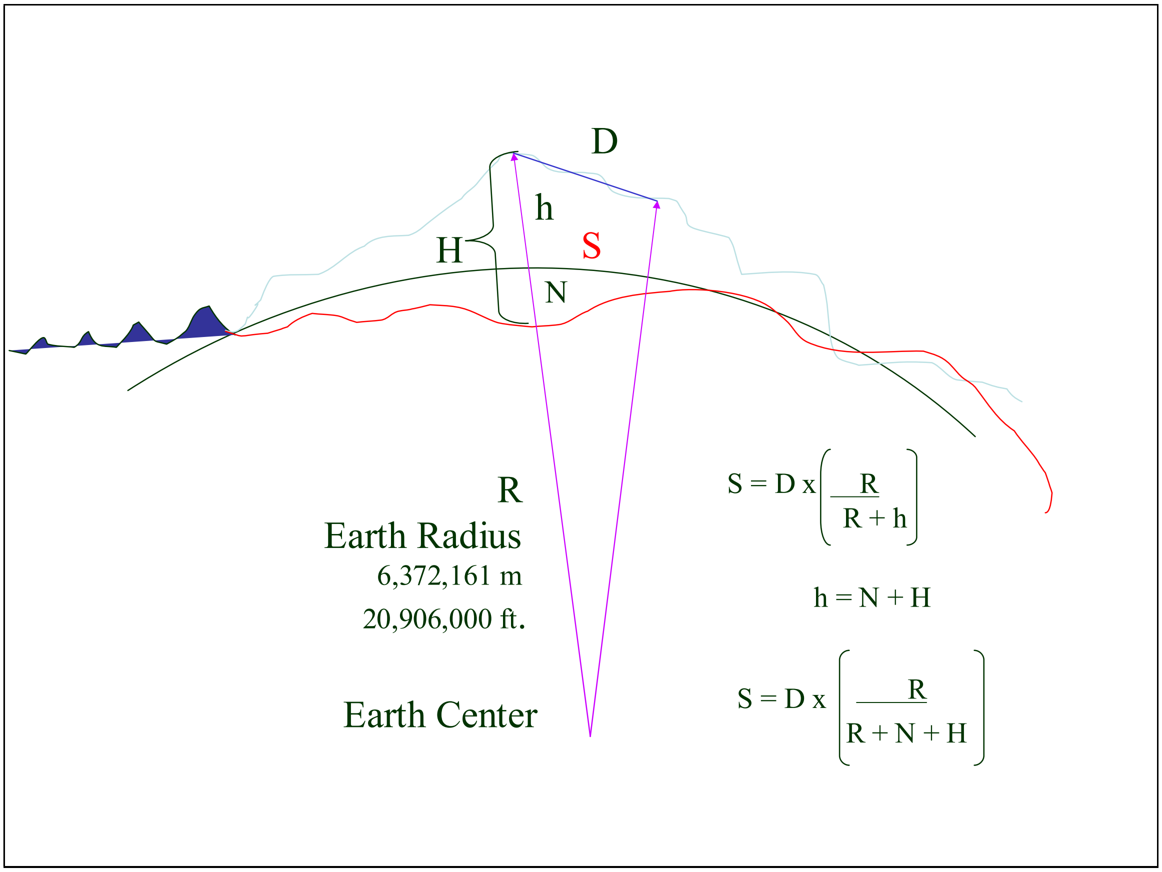

Figure 5-9. Reduction of measured slope distance D to ellipsoid distance S (NGS)

a. Grid factor. For most topographic surveys covering a small geographical site, these two factors can be combined into a constant "grid factor"--reference Kavanagh 1997:

Grid factor = Sea Level Factor x Scale Factor

then: *Ground Distance = Grid Distance / Grid Factor*

or

Grid Distance = Ground Distance x Grid Factor

b. Convergence. Between two fixed points, the geodetic azimuth will differ from the grid azimuth. This difference is known as "convergence" and varies with the distance from the central meridian of the projection. Thus, if a geodetic azimuth is given between two fixed points (inversed from published geographic coordinates, astronomic, or GPS), then it must be corrected for convergence to obtain an equivalent grid azimuth. If lengthy control traverses are being computed on a SPCS or UTM grid, then additional second term corrections to observed angles may be required--e.g., the "t-T" correction used in older survey manuals at Appendix A-4 (TM 5-237 and TM 5-241-2).

c. Use of data collectors. The above grid corrections should rarely have to be performed when modern survey data collectors are being used. These total station or RTK data collectors (with full COGO and adjustment capabilities) will automatically perform all the necessary geographic to grid coordinate translations, including sea level reductions and local grid system conversions that are later transformed and adjusted into an established SPCS grid at a true elevation. If for some reason you are not using a data collector that seamlessly performs these translation functions, and you are performing a survey in higher elevations, then you must correct original distance observations for the sea level reduction. If you transfer these observed distances and angles to a SPCS or UTM grid, then you must correct for grid scale factor and convergence described above. Consult any of the referenced surveying textbooks at Appendix A-2 for procedures and examples.

5-11. Universal Transverse Mercator Coordinate System

Universal Transverse Mercator (UTM) coordinates are used in surveying and mapping when the size of the project extends through several state plane zones or projections. UTM coordinates are also utilized by the US Army, Air Force, and Navy for tactical mapping, charting, and geodetic applications. It may also be used to reference site plan engineering surveys if so requested in CONUS or OCONUS installations. The UTM projection differs from the TM projection in the scale at the central meridian, origin, and unit representation. The scale at the central meridian of the UTM projection is 0.9996. In the Northern Hemisphere, the northing coordinate has an origin of zero at the equator. In the Southern Hemisphere, the southing coordinate has an origin of ten million meters (10,000,000 m). The easting coordinate has an origin five hundred thousand meters (500,000 m) at the central meridian. The UTM system is divided into sixty (60) longitudinal zones. Each zone is six (6) degrees in width extending three (3) degrees on each side of the central meridian. UTM coordinates are always expressed in meters. USACE program CORPSCON can be used to transform coordinates between UTM and SPCS systems. Additional details on UTM grids and survey computations thereon may be found in the older DA references listed at Appendix A-4.

5-12. The US Military Grid-Reference System (FM 3-34.331)

The US Military Grid-Reference System (MGRS) is designed for use with UTM grids. For convenience, the earth is generally divided into 6° by 8° geographic areas, each of which is given a unique grid-zone designation. These areas are covered by a pattern of 100,000-meter squares. Two letters (called the 100,000-meter-square letter identification) identify each square. This identification is unique within the area covered by the grid-zone designation.

a. The MGRS is an alphanumeric version of a numerical UTM grid coordinate. Thus, for that portion of the world where the UTM grid is specified (80° south to 84° north), the UTM grid-zone number is the first element of a military grid reference. This number sets the zone longitude limits. The next element is a letter that designates a latitude bond. Beginning at 80° south and proceeding northward, 20 bands are lettered C through X. In the UTM portion of the MGRS, the first three characters designate one of the areas within the zone dimensions.

b. A reference that is keyed to a gridded map (of any scale) is made by giving the 100,000-meter-square letter identification together with the numerical location. Numerical references within the

100,000-meter square are given to the desired accuracy in terms of the easting and northing grid coordinates for the point.

c. The final MGRS position coordinate consists of a group of letters and numbers that include the following elements:

- The grid-zone designation.
- The 100,000-meter-square letter identification.
- The grid coordinates (also referred to as rectangular coordinates) of the numerical portion of the reference, expressed to a desired refinement.
- The reference is written as an entity without spaces, parentheses, dashes, or decimal points.

Examples are as follows:

18S (locating a point within the grid-zone designation).
18SUU (locating a point within a 100,000-meter square).
18SUU80 (locating a point within a 10,000-meter square).
18SUU8401 (locating a point within a 1,000-meter square).
18SUU836014 (locating a point within a 100-meter square).

d. To satisfy special needs, a reference can be given to a 10-meter square and a 1-meter square. Examples are as follows:

8SUU83630143 (locating a point within a 10-meter square).
18SUU8362601432 (locating a point within a 1-meter square).

e. There is no zone number in the polar regions. A single letter designates the semicircular area and the hemisphere. The letters A, B, Y, and Z are used only in the polar regions. An effort is being made to reduce the complexity of grid reference systems by standardizing a single, worldwide grid reference system.

5-13. US National Grid System

A US National Grid (USNG) system has been developed to improve public safety, commerce, and aid the casual GPS user with an easy to use geoaddress system for identifying and determining location with the help of a USNG gridded map and/or a USNG enabled GPS system. The USNG can provide for whatever level of precision is desired. Many users may prefer to continue using the UTM format for applications requiring precision greater than 1 meter.

a. Grid Zone Designation (GZD). The US geographic area is divided into 6-degree longitudinal zones designated by a number and 8-degree latitudinal bands designated by a letter. Each area is given a unique alphanumeric Grid Zone Designator--e.g., 18S.

b. 100,000-meter square identification. Each GZD 6x8 degree area is covered by a specific scheme of 100,000-meter squares where each square is identified by two unique letters--e.g., 18SUJ identifies a specific 100,000-meter square in the specified GZD.

c. Grid coordinates. A point position within the 100,000-meter square shall be given by the UTM grid coordinates in terms of its Easting (E) and Northing (N). An equal number of digits shall be used for

E and N where the number of digits depends on the precision desired in position referencing. In this convention, the reading shall be from left with Easting first and then Northing.

Examples:

18SUJ20 - Locates a point with a precision of 10 km

18SUJ2306 - Locates a point with a precision of 1 km

18SUJ234064 - Locates a point with a precision of 100 meters

18SUJ23480647 - Locates a point with a precision of 10 meters

18SUJ2348306479 - Locates a point with a precision of 1 meter

The number of digits in Easting and Northing can vary, depending on specific requirements or application.

5-14. Chainage-Offset Coordinate Systems

Most linear engineering and construction projects (roads, railways, canals, navigation channels, levees, floodwalls, beach renourishment, etc.) are locally referenced using the traditional engineering chainage-offset system--Figure 5-10. Usually, SPCS coordinates are provided at the PIs, from which (given the alignment between PIs) a SPCS coordinate can then be computed for any given station-offset point. Chainage-offset systems are used for locating cross-sections along even centerline stations. Topographic elevation and feature data is then collected along each section relative to the centerline. Likewise, road, canal, levee alignments can be staked out relative to station-offset parameters, and internally in a total station or RTK system data collector, these offsets may actually be transformed from a SPCS.

a. Station. Alignment stationing (or chainage) zero references are arbitrarily established for a given project or sectional area. Stationing on a navigation project usually commences offshore on coastal projects and runs inland or upstream. Stationing follows the channel centerline alignment. Stationing may be accumulated through each PI or zero out at each PI or new channel reach. Separate stationing is established for widener sections, turning basins, levees, floodwalls, etc. Each district may have its own convention. Stationing coordinates use "+" signs to separate the second- and third-place units (XXX + XX.XX). Metric chainage often separates the third and fourth places (XXX + XXX.XX) to distinguish the units from English feet; however, some districts use this convention for English stationing units.

b. Offsets. Offset coordinates are distances from the centerline alignment. Offsets carry plus/minus coordinate values. Normally, offsets are positive to the right (looking toward increasing stationing). Some USACE Districts designate cardinal compass points (east-west or north-south) in lieu of a coordinate sign. On some navigation projects, the offset coordinate is termed a "range," and is defined relative to the project centerline or, in some instances, the channel-slope intersection line (toe). Channel or canal offsets may be defined relative to a fixed baseline on the bank or levee.

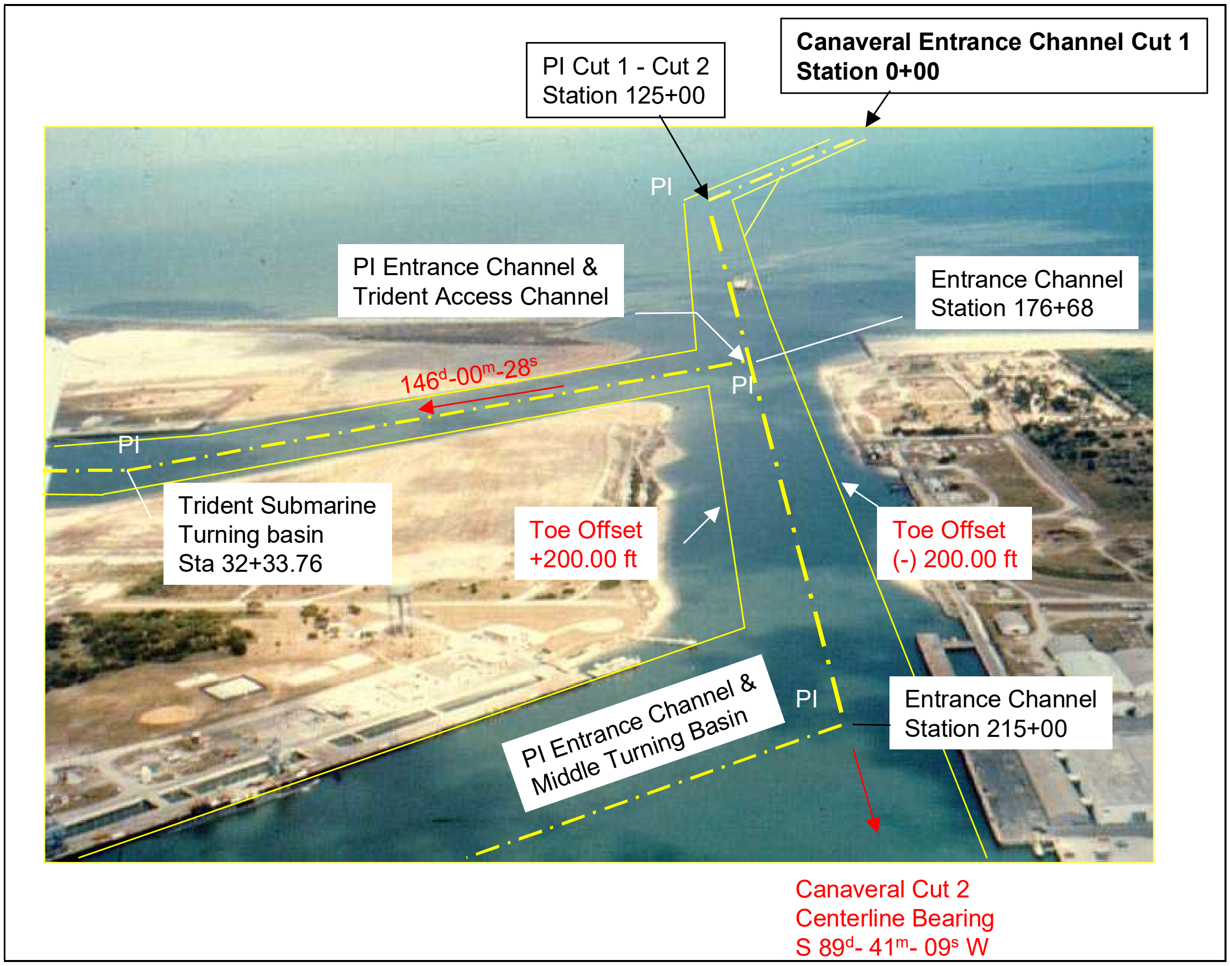

Figure 5-10. Chainage-offset project control scheme for a typical deep-draft navigation project--Cape Canaveral, FL (Jacksonville District)

c. Azimuth. Azimuths are computed relative to the two defining PIs. Either 360-deg azimuth or bearing designations may be used. Azimuths should be shown to the nearest second.

d. Other local alignments. Different station-offset reference grids may be established for individual portions of a project. River sections and coastal beach sections are often aligned perpendicular to the project/coast. Each of these sections is basically a separate local datum with a different reference point and azimuth alignment. Beach sections may also be referenced to an established coastal construction setback line. Circular and transition (spiral) curve alignments are also found in some rivers, canals, and flood control projects such as spillways and levees. Surveys will generally be aligned to the chainage and offsets along such curves. Along inland waterways, such as the Mississippi River, stationing is often referenced to either arbitrary or monumented baselines along the bank. In many instances, a reference baseline for a levee is used, and surveys for revetment design and construction are performed from offsets to this line. Separate baselines may exist over the same section of river, often from levees on opposite banks or as the result of revised river flow alignments. Baseline stationing may increase either upstream or downstream. Most often, the mouth of a river is considered the starting point (Station 0 + 00), or the river reaches are summed to assign a station number at the channel confluence. Stationing may increase consecutively through PIs or reinitialize at channel turns. In addition, supplemental horizontal reference may also be made to a river mile designation system. River mile systems established years ago may no longer be exact if the river course has subsequently realigned itself. River mile designations can be used to specify geographical features and provide navigation reference for users.

5-15. Datum Conversions and Transformation Methods

a. General. Federal Geodetic Control Subcommittee (FGCS) members, which includes USACE, have adopted NAD 83 as the standard horizontal datum for surveying and mapping activities performed or financed by the Federal government. To the extent practicable, legally allowable, and feasible, USACE should use NAD 83 in its surveying and mapping activities. Transformations between NAD 27 coordinates and NAD 83 coordinates are generally obtained using the CORPS Convert (CORPSCON) software package or other North American Datum Conversion (e.g., NADCON) based programs.

b. Conversion techniques. USACE survey control published in the NGS control point database has been already converted to NAD 83 values. However, most USACE survey control was not originally in the NGS database and was not included in the NGS readjustment and redefinition of the national geodetic network. Therefore, USACE will have to convert this control to NAD 83. Coordinate conversion methods considered applicable to USACE projects are discussed below.

(1) Resurvey from NAD 83 Control. A new survey using NGS published NAD 83 control could be performed over the entire project. This could be either a newly authorized project or one undergoing major renovation or maintenance. Resurvey of an existing project must tie into all monumented points. Although this is not a datum transformation technique, and would not normally be economically justified unless major renovation work is being performed, it can be used if existing NAD 27 control is of low density or accuracy.

(2) Readjustment of Survey. If the original project control survey was connected to NGS control stations, the survey may be readjusted using the NAD 83 coordinates instead of the NAD 27 coordinates originally used. This method involves locating the original field notes and observations, and completely readjusting the survey and fixing the published NAD 83 control coordinates.

(3) Mathematical Transformations. Since neither of the above methods can be economically justified on most USACE projects, mathematical approximation techniques for transforming project control data to NAD 83 have been developed. These methods yield results which are normally within ± 1 ft of the actual values and the distribution of errors are usually consistent within a local project area. Since these coordinate transformation techniques involve approximations, they should be used with caution when real property demarcation points and precise surveying projects are involved. When mathematical transformations are employed they should be adequately noted so that users will be aware of the conversion method.

c. Horizontal datum transformation methods. Coordinate transformations from one geodetic reference system to another can be most practically made either by using a local seven-parameter transformation or by interpolation of datum shift values across a given region.

(1) Seven parameter transformations. For worldwide (OCONUS) and local datum transformations, one of the referenced textbooks at Section A-2 should be consulted.

(2) Grid-shift transformations. Current methods for interpolation of datum shift values use the difference between known coordinates of common points from both the NAD 27 and NAD 83 adjustments to model a best-fit shift in the regions surrounding common points. A grid of approximate

datum shift values is established based on the computed shift values at common points in the geodetic network. The datum shift values of an unknown point within a given grid square are interpolated along each axis to compute an approximate shift value between NAD 27 and NAD 83. Any point that has been converted by such a transformation method should be considered as having only approximate NAD 83 coordinates.

(3) NADCON/CORPSCON. NGS developed the transformation program NADCON, which yields consistent NAD 27 to NAD 83 coordinate transformation results over a regional area. This technique is based on the above grid-shift interpolation approximation. NADCON was reconfigured into a more comprehensive program called CORPSCON. Technical documentation and operating instructions for CORPSCON can be obtained at the following ERDC Web site: http://crunch.tec.army.mil. This software converts between:

NAD 27	NAD 83	SPCS 27	SPCS 83
UTM 27	UTM 83	NGVD 29	NAVD 88
GEOID03	HARN		

Since the overall CORPSCON datum shift (from point to point) varies throughout North America, the amount of datum shift across a local project is also not constant. The variation can be as much as 0.1 ft per mile. Examples of some NAD 27 to NAD 83 based coordinate shift variations that can be expected over a 10,000-ft section of a project are shown below:

Project Area	SPCS Reference	Per 10,000 feet
Baltimore, MD	1900	0.16 ft
Los Angeles, CA	0405	0.15 ft
Mississippi Gulf Coast	2301	0.08 ft
Mississippi River (IL)	1202	0.12 ft
New Orleans, LA	1702	0.22 ft
Norfolk, VA	4502	0.08 ft
San Francisco, CA	0402	0.12 ft
Savannah, GA	1001	0.12 ft
Seattle, WA	4601	0.10 ft

Such local scale changes will cause project alignment data to distort by unequal amounts. Thus, a 10,000-ft tangent on NAD 27 project coordinates could end up as 9,999.91 feet after mathematical transformation to NAD 83 coordinates. Although such differences may not be appear significant from a lower-order construction survey standpoint, the potential for such errors must be recognized. Therefore, the transformations will not only significantly change absolute coordinates on a project and the datum transformation process will slightly modify the project's design dimensions and/or construction orientation and scale. For example, on a navigation project, an 800.00-ft wide channel could vary from 799.98 to 800.04 feet along its reach. This variation could also affect grid alignment azimuths. Moreover, if the local SPCS 83 grid was further modified, then even larger dimension changes can result. Correcting for distortions may require recomputation of coordinates after conversion to ensure original project dimensions and alignment data remain intact. This is particularly important for property and boundary surveys. A less accurate alternative is to compute a fixed shift to be applied to all data points over a limited area. Determining the maximum area over which such a fixed shift can be applied is important. Computing a fixed conversion factor with CORPSCON can be made to within ±1 foot. Typically, this fixed conversion would be computed at the center of a sheet or at the center of a project and the conversions in X and Y from NAD 27 to NAD 83 and from SPCS 27 to SPCS 83 indicated by notes on the sheets or data sets. Since the conversion is not constant over a given area, the fixed conversion amounts must be explained in the note. The magnitude of the conversion factor change across a sheet is a function of location and the drawing scale. Whether the magnitude of the distortion is significant depends on the nature of the project. For example, a 0.5-ft variation on an offshore navigation project may be acceptable for converting depth sounding locations, whereas a 0.1-ft change may be intolerable for construction layout on an installation. In any event, the magnitude of this gradient should be computed by CORPSCON at each end (or corners) of a sheet or project. If the conversion factor variation exceeds the allowable tolerances, then a fixed conversion factor should not be used. Two examples of using Fixed Conversion Factors follow:

Example 1. Assume a 1 inch = 40 ft scale site plan map on existing SPCS 27 (VA South Zone 4502). Using CORPSCON, convert existing SPCS 27 coordinates at the sheet center and corners to SPCS 83 (US Survey Foot), and compare SPCS 83-27 differences.

SPCS 83	SPCS 27	SPCS 83 - SPCS 27	
Center	N 3,527,095.554	Y 246,200.000	dY = 3,280,895.554
of Sheet	E 11,921,022.711	X 2,438,025.000	dX = 9,482,997.711
NW	N 3,527,595.553	Y 246,700.000	dY = 3,280,895.553
Corner	E 11,920,522.693	X 2,437,525.000	dX = 9,482,997.693
NE	N 3,527,595.556	Y 246,700.000	dY = 3,280,895.556
Corner	E 11,921,522.691	X 2,438,525.000	dX = 9,482,997.691
SE	N 3,526,595.535	Y 245,700.000	dY = 3,280,895.535
Corner	E 11,921,522.702	X 2,438,525.000	dX = 9,482,997.702
SW	N 3,526,595.535	Y 245,700.000	dY = 3,280,895.535
Corner	E 11,920,522.704	X 2,437,525.000	dX = 9,482,997.704

Since coordinate differences do not exceed 0.03 feet in either the X or Y direction, the computed SPCS 83-27 coordinate differences at the center of the sheet may be used as a fixed conversion factor to be applied to all existing SPCS 27 coordinates on this drawing.

Example 2. Assuming a 1inch = 1,000 ft base map is prepared of the same general area, a standard drawing will cover some 30,000 feet in an east-west direction. Computing SPCS 83-27 differences along this alignment yields the following:

SPCS 83	SPCS 27	SPCS 83 - SPCS 27	
West	N 3,527,095.554	Y 246,200.000	dY = 3,280,895.554
End	E 11,921,022.711	X 2,438,025.000	dX = 9,482,997.711
East	N 3,527,095.364	Y 246,200.000	dY = 3,280,895.364
End	E 11,951,022.104	X 2,468,025.000	dX = 9,482,997.104

The conversion factor gradient across this sheet is about 0.2 ft in Y and 0.6 ft in X. Such small changes are not significant at the plot scale of 1 inch = 1,000 ft; however, for referencing basic design or construction control, applying a fixed shift across an area of this size is not recommended -- individual points should be transformed separately. If this 30,000-ft distance were a navigation project, then a fixed conversion factor computed at the center of the sheet would suffice for all bathymetric features. Caution should be exercised when converting portions of projects or military installations or projects that are adjacent to other projects that may not be converted. If the same monumented control points are used for several projects or parts of the same project, different datums for the two projects or parts thereof could lead to surveying and mapping errors, misalignment at the junctions and layout problems during construction.

d. Dual grids ticks. Depicting both NAD 27 and NAD 83 grid ticks and coordinate systems on maps and drawings should be avoided where possible. This is often confusing and can increase the chance for errors during design and construction. However, where use of dual grid ticks and coordinate systems is unavoidable, only secondary grid ticks in the margins will be permitted.

e. Field survey methods. If GPS is used to set new control points referenced to higher order control many miles from the project (e.g., CORS networks), inconsistent data may result at the project site. If the new control is near older control points that have been converted to NAD 83 using CORPSCON, two slightly different network solutions can result, even though both have NAD 83 coordinates. In order to avoid these situations, it is recommended that all project control (old and new) be tied into the same reference system--preferably the NSRS.

f. Local project datums. Local project datums that are not referenced to NAD 27 cannot be mathematically converted to NAD 83 with CORPSCON. Field surveys connecting them to other stations that are referenced to NAD 83 are required.

5-16. Horizontal Transition Plan from NAD 27 to NAD 83

a. General. Not all maps, engineering site drawings, documents, and associated products containing coordinate information will require conversion to NAD 83. To insure an orderly and timely transition to NAD 83 is achieved for the appropriate products, the following general guidelines should be followed:

(1) Initial surveys. All initial surveys should be referenced to NAD 83.

(2) Active projects. Active projects where maps, site drawings or coordinate information are provided to non-USACE users (e.g., NOAA, USCG, FEMA, and others in the public and private sector) coordinates should be converted to NAD 83 the next time the project is surveyed or maps or site drawings are updated for other reasons.

(3) Inactive projects. For inactive projects or active projects where maps, site drawings or coordinate information are not normally provided to non-USACE users, conversion to NAD 83 is optional.

(4) Datum notes. Whenever maps, site drawings or coordinate information (regardless of type) are provided to non-USACE users, it should contain a datum note, such as the following:

> THE COORDINATES SHOWN ARE REFERENCED TO NAD *[27/83] AND ARE IN FEET BASED ON THE SPCS *[27/83] *[STATE, ZONE]. DIFFERENCES BETWEEN NAD 27 AND NAD 83 AT THE CENTER OF THE *[SHEET/DATASET] ARE *[dLat, dLon, dX, dY]. DATUM CONVERSION WAS PERFORMED USING THE COMPUTER PROGRAM “CORPSCON.” METRIC CONVERSIONS WERE BASED ON THE *[US SURVEY FOOT = 1200/3937 METER] [INTERNATIONAL FOOT = 30.48/100 METER].

b. Levels of effort. For maps and site drawings the conversion process entails one of three levels of effort:

(1) Conversion of coordinates of all mapped details to NAD 83, and redrawing the map,

(2) Replace the existing map grid with a NAD 83 grid,

(3) Simply adding a datum note.

For surveyed points, control stations, alignment, and other coordinated information, conversion must be made either through a mathematical transformation or through readjustment of survey observations.

c. Detailed instructions.

(1) Initial surveys on Civil Works projects. The project control should be established on NAD 83 relative to NGS's National Spatial Reference System (NSRS) using conventional or GPS surveying procedures. The local SPCS 83 grid should be used on all maps and site drawings. All planning and design activities should then be based on the SPCS 83 grid. This includes supplemental site plan mapping, core borings, project design and alignment, construction layout and payment surveys, and applicable boundary or property surveys. All maps and site drawings shall contain datum notes. If the local sponsor requires the use of NAD 27 for continuity with other projects that have not yet converted to NAD 83, conversion to NAD 27 could be performed using the CORPSCON transformation techniques.

(2) Active Civil Works Operations and Maintenance projects undergoing maintenance or repair. These projects should be converted to NAD 83 during the next maintenance or repair cycle in the same manner as for newly initiated civil works projects. However, if resources are not available for this level of effort, either redraw the grids or add the necessary datum notes. Plans should be made for the full conversion during a later maintenance or repair cycle when resources can be made available.

(3) Military Construction and master planning projects. All installations and master planning projects should remain on NAD 27 or the current local datum until a thoroughly coordinated effort can be arranged with the MACOM and installation. An entire installation's control network should be transformed simultaneously to avoid different datums on the same installation. The respective MACOMs are responsible for this decision. However, military operations may require NAD 83, including SPCS 83 or UTM metric grid systems. If so, these shall be performed separate from facility engineering support. A dual grid system may be required for such operational applications when there is overlap with normal facilities engineering functions. Coordinate transformations throughout an installation can be computed using the procedures described herein. Care must be taken when using transformations from NAD 27 with new control set using GPS methods from points remote from the installation. Installation boundary surveys should adhere to those outlined under real estate surveys listed below.

(4) Real Estate. Surveys, maps, and plats prepared in support of civil works and military real estate activities should conform as much as possible to state requirements. Since most states have adopted NAD 83, most new boundary and property surveys should be based on NAD 83. The local authorities should be contacted before conducting boundary and property surveys to ascertain their policies. It should be noted that several states have adopted the International Foot for their standard conversion from meters to feet. In order to avoid dual coordinates on USACE survey control points that have multiple uses, all control should be based on the US Survey Foot, including control for boundary and property surveys. In states where the International Foot is the only accepted standard for boundary and property surveys, conversion of these points to NAD 83 should be based on the International foot, while the control remains based on the US Survey foot.

(5) Regulatory functions. Surveys, maps, and site drawings prepared in support of regulatory functions should begin to be referenced to NAD 83 unless there is some compelling reason to remain on NAD 27 or locally used datum. Conversion of existing surveys, maps, and drawings to NAD 83 is not necessary. Existing surveys, maps, and drawings need only have the datum note added before distribution to non-USACE users. The requirements of local, state and other Federal permitting agencies should be ascertained before site specific conversions are undertaken. If states require conversions based on the International foot, the same procedures as described above for Real Estate surveys should be followed.

(6) Other existing projects. Other existing projects, e.g., beach nourishment, submerged offshore disposal areas, historical preservation projects, etc., need not be converted to NAD 83. However, existing surveys, maps, and drawings should have the datum note added before distribution to non-USACE users.

(7) Work for others. Existing projects for other agencies will remain on NAD 27 or the current local datum until a thoroughly coordinated effort can be arranged with the sponsoring agency. The decision to convert rests with the sponsoring agency. However, existing surveys, maps, and drawings should have the datum note added before distribution to non-USACE users. If sponsoring agencies do not indicate a preference for new projects, NAD 83 should be used. The same procedures as described above for initial surveys on Civil Works projects should be followed.

SECTION III
Vertical Reference Systems

A vertical datum is the surface to which elevations or depths are referred to or referenced. There are many vertical datums used within CONUS. The surveyor should be aware of the vertical control datum being used and its practicability to meet project requirements. .

5-17. National Geodetic Vertical Datum of 1929 (NGVD 29)

NGVD 29 was established by the United States Coast and Geodetic Survey (USC&GS) 1929 General Adjustment by constraining the combined US and Canadian First Order leveling nets to conform to Mean Sea Level (MSL). It was determined at 26 long-term tidal gage stations that were spaced along the east and west coast of North American and along the Gulf of Mexico, with 21 stations in the US and 5 stations in Canada. NGVD 29 was originally named the Mean Sea Level Datum of 1929. It was known at the time that the MSL determinations at the tide gages would not define a single equipotential surface because of the variation of ocean currents, prevailing winds, barometric pressures, and other physical causes. The name of the datum was changed from the Mean Sea Level Datum to the NGVD 29 in 1973 to eliminate the reference to sea level in the title. This was a change in name only; the definition of the datum established in 1929 was not changed. Since NGVD 29 was established, it has become obvious that the geoid based upon local mean tidal observations would change with each measurement cycle. Estimating the geoid based upon the constantly changing tides does not provide a stable estimate of the shape of the geoid.

5-18. 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. Most Third Order benchmarks, including those of other Federal, state and local government agencies, were not included in the NAVD 88 adjustment. 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 5-11 below. Therefore, it is important that these two reference systems not be confused. Given the local variations shown in Figure 5-14, there is no direct transformation between the two systems, and a site calibration/transformation must be performed as explained in subsequent sections.

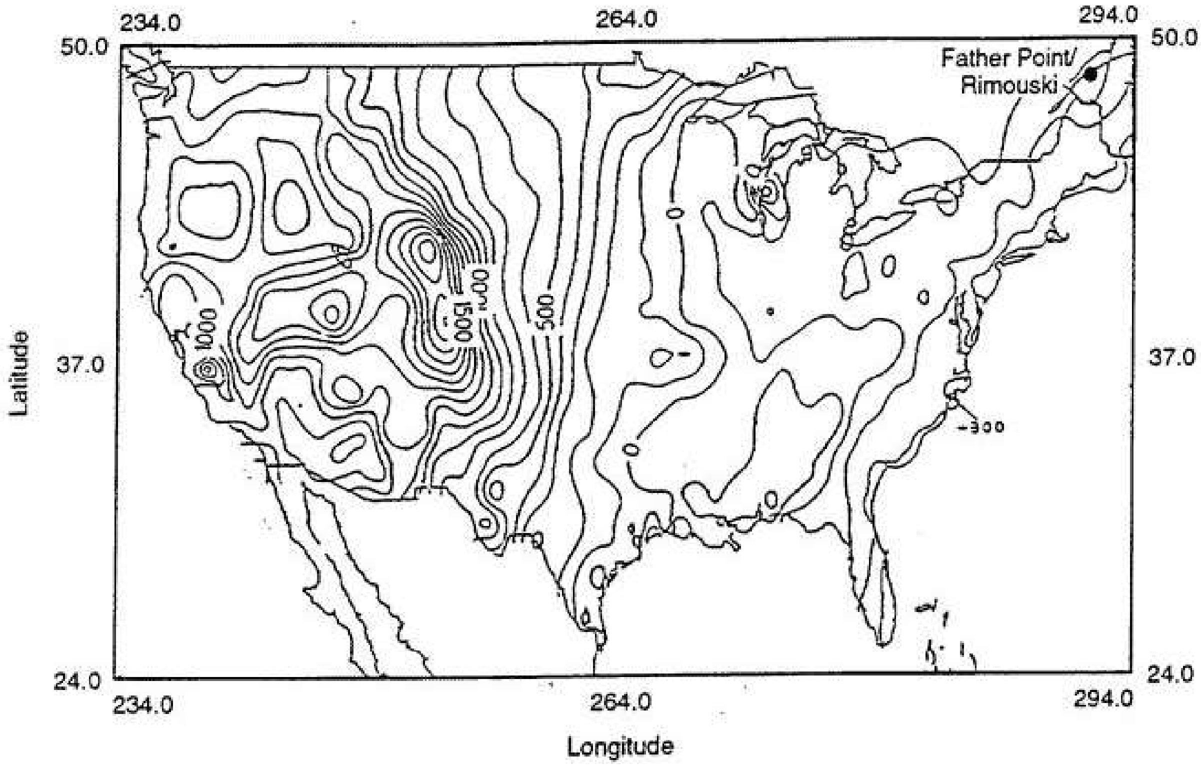

Figure 5-11. NGVD 29-NAVD 88 elevation differences in mm

The Federal Geodetic Control Subcommittee (FGCS) of the Federal Geographic Data Committee (FGDC) has affirmed that NAVD 88 shall be the official vertical reference datum for the US. The FGDC has prescribed that all surveying and mapping activities performed or financed by the Federal Government make every effort to begin an orderly transition to NAVD 88, where practicable and feasible. Further technical details on NAVD 88 are in Appendix C, "*Development and Implementation of NAVD 88*."

5-19. Other Vertical Reference Datums and Planes

a. Mean Sea Level datums. Some vertical datums are referenced to mean seal level. Such datums typically are maintained locally or within a specific project area. The theoretical basis for these datums is local mean sea level. Local MSL is a vertical datum based on observations from one or more tidal gaging stations. NGVD 29 was based upon the assumption that local MSL at 21 tidal stations in the US and 5 tidal stations in Canada averaged 0.0 ft on NGVD 29. The value of MSL as measured over the Metonic cycle of 19 years shows that this assumption is not valid and that MSL varies from station to station.

b. Great Lake datums. Depths in the Great Lakes and connecting channels are referenced to the International Great Lakes Datum (IGLD) of 1985. IGLD 85 represents a low water datum from which navigation is maintained. A separate datum is established for each of the Great Lakes. The datum must be adjusted for slope in the connecting channels between the Great Lakes. These datums undergo periodic adjustment. For example, the IGLD 55 was adjusted in 1985 to produce IGLD 85. IGLD 85 has been directly referenced to NAVD 88 and originates at the same point as NAVD 88. Additional details are provided in Appendix C.

c. Other vertical datums. Other areas may maintain and employ specialized vertical datums. For instance, vertical datums maintained in Alaska, Puerto Rico, Hawaii, the Virgin Islands, Guam, and other islands and project areas. Specifications and other information for these particular vertical datums can be obtained from the particular District responsible for survey related activities in these areas, or the National Ocean Service (NOS).

d. Tidal areas. Tidal datums usually are defined by the range and phase of the tide and usually are referenced to a mean lower low water elevation, or MLLW. In offshore coastal areas, CONUS navigation projects are generally referenced to a MLLW datum established by NOS or the Corps from long-term gage observations. This MLLW reference plane is not a flat surface but slopes as a function of the tidal range in the area. Tidal range can increase or decrease near coastal entrances; thus the MLLW must be accurately modeled throughout the navigation project. The required grade at all points on the navigation project is dependent on tidal modeling--requiring determination of the elevation of the MLLW datum plane from a series of gage observations at each point. For further information on these and other tidal datum related terms, refer to Appendix B, "*Requirements and Procedures for Referencing Coastal Navigation Projects to Mean Lower Low Water (MLLW) Datum*" and EM 1110-2-1003 (*Hydrographic Surveying*)

e. Inland river areas. River datums are usually referenced to a low water reference plane (LWRP), such as the LWRP 1974 reference used in the unregulated portion of the Mississippi River. Like tidal MLLW, the low water river datum must be determined from gage/staff observations at sufficient points along the river to adequately define the surface. The spacing of these observations must be sufficient to allow linear interpolation between staff gage points. For a river like the Mississippi that drops 0.5 ft/mile, gages or benchmarks may be required at least every quarter- to half-mile in order to reference hydrographic surveys.

f. Controlled river pools. Between river control structures, low water pools are used to reference maintained navigation depths. Since these pools themselves may exhibit some slope, sufficient gages/benchmarks within the pools should be established to account for any minor slope.

g. Reservoir pools. Depths in controlled reservoirs are usually referenced to a national vertical datum (e.g., NGVD 29 or NAVD 88).

5-20. Orthometric Elevations

Orthometric elevations are those corresponding to the earth's irregular geoidal surface, as illustrated in Figure 5-12 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, which approximates mean sea level near coastal points. 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 (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 NGVD 29 or the updated NAVD 88. The NGVD 29 reference datum more closely 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. GPS derived ellipsoidal heights shown in Figure 5-12 below must be converted to local orthometric elevations in order to have useful engineering and construction value--see EM 1110-1-1003 for details. 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.

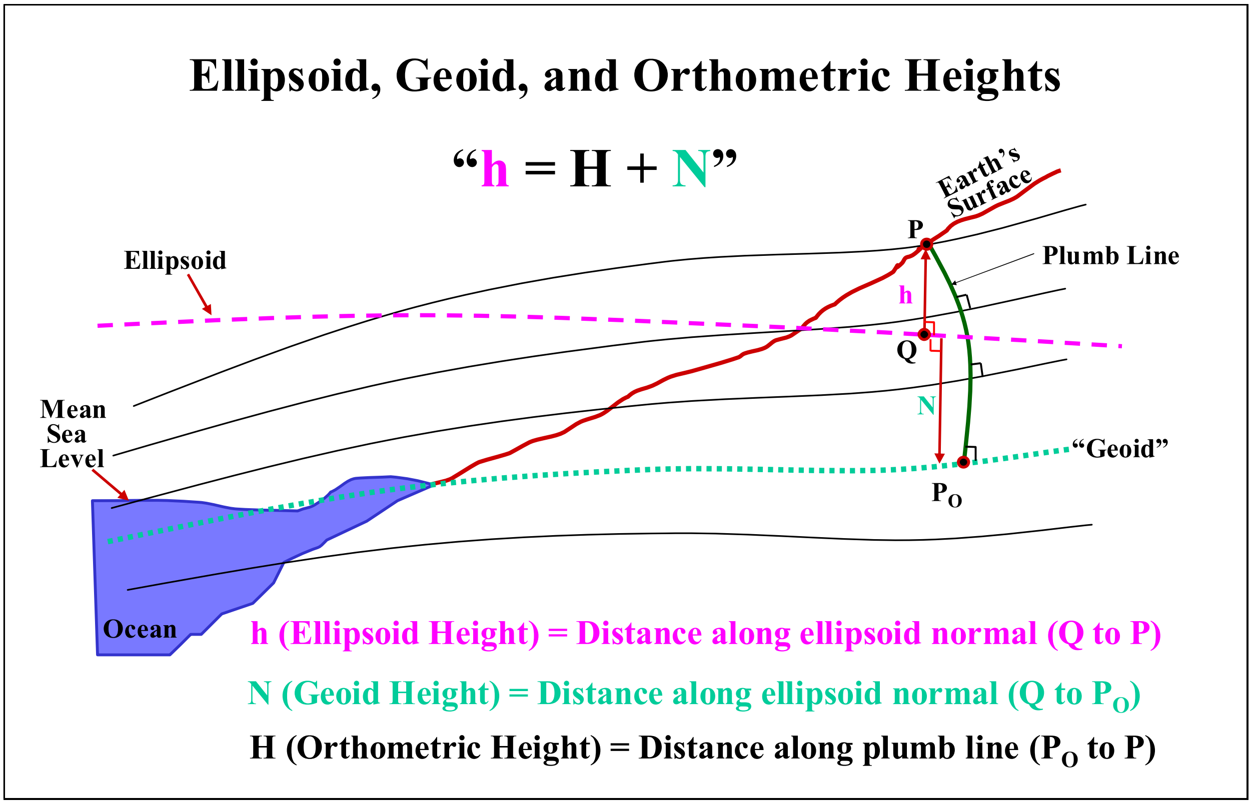

Figure 5-12. Ellipsoid, geoid, and orthometric surface definitions and relationships (NGS)

5-21. WGS 84 Ellipsoidal Heights

GPS-determined heights (or height differences) are referenced to an idealized mathematical ellipsoid which differs significantly from the geoid; thus, GPS heights are not the same as the orthometric heights needed for standard USACE projects (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, at greater distances, a site calibration with surrounding benchmarks must be performed in order to adjust RTK ellipsoidal heights down to the local vertical datum. For some surveys (e.g., offshore navigation), a predicted geoid model may be used if no other vertical control is available to calibrate the model.

5-22. Orthometric Height and WGS 84 Ellipsoidal Elevation Relationship

Geoidal heights represent the geoid-ellipsoid separation distance 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, and depicted graphically in Figure 5-12 above.

$$h = H + N \qquad \text{(Eq 5-2)}$$

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 above the ellipsoid.

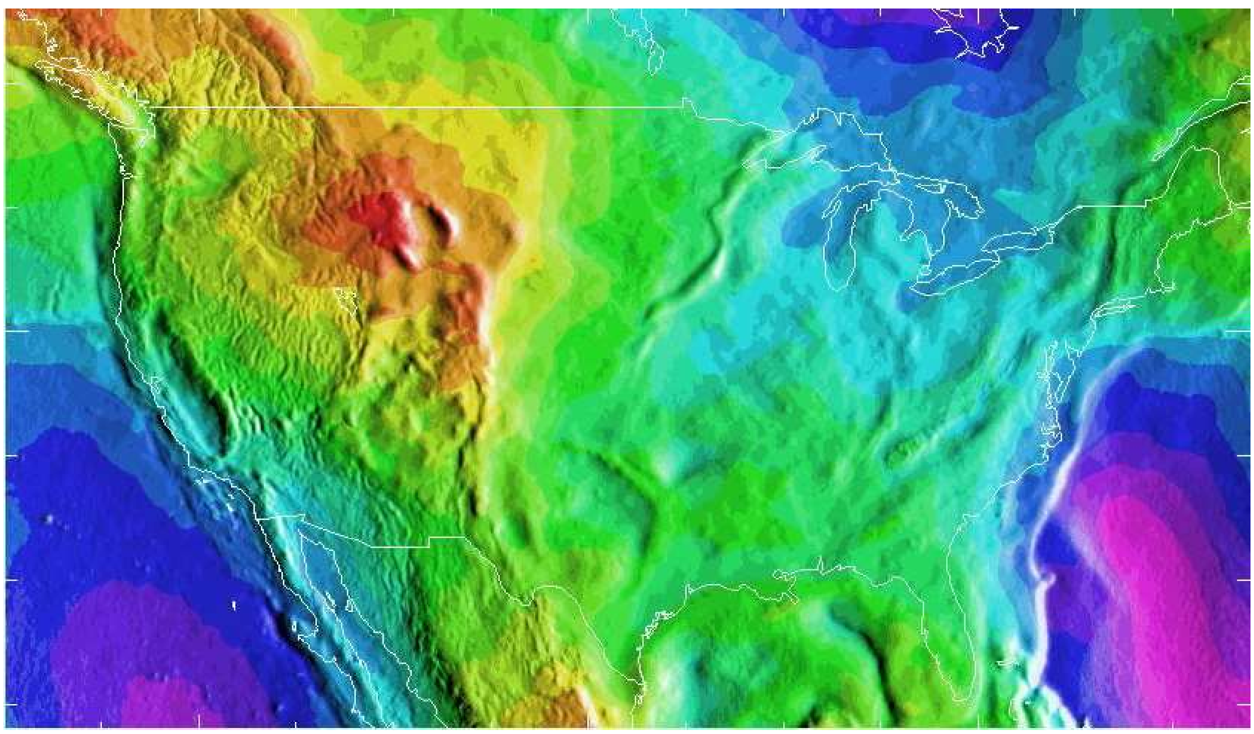

Figure 5-13. Geoid undulation model of North America--depicts geoid undulation *N* relative to the WGS 84 ellipsoid

5-23. 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 5-13 above. Geoid modeling techniques are used to obtain the parameter *"N"* in the above equation from which ellipsoidal heights can be converted to orthometric elevations. These geoid models (e.g., Geoid 90, Geoid 93, Geoid 96, Geoid 99, Geoid 03, 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 relatively 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 which 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 absolute accuracy for geoid heights to a few centimeters.

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 (e.g., 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 in Appendix A or the National Geodetic Survey web site.

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 accurate (i.e., ± 0.01 ft) construction surveying purposes; however, GPS-derived elevations may be adequate for small-scale topographic mapping control and hydrographic surveying applications where ± 0.1 ft accuracy is sufficient.

5-24. 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 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 unknown 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.

5-25. Vertical Datum Transformations

Appendix C (*Development and Implementation of NAVD 88)* contains a detailed discussion on the development and implementation of NAVD 88, and the rationale for converting projects from NGVD 29 to this updated vertical datum. There are several reasons for USACE commands to convert authorized and future projects to NAVD 88--these are summarized from Appendix C.

- Differential leveling surveys will close better.

- NAVD 88 height values are available in convenient form from the NGS database.

- Federal surveying and mapping agencies will stop publishing on NGVD 29.

- NAVD 88 is recommended by ACSM and FGCS.

- Surveys performed for the Federal government require use of NAVD 88.

- NAVD 88 provides a reference to estimate GPS derived orthometric heights.

The last bullet above is a primary reason for transforming project control to NAVD 88. The conversion process entails one of two levels of effort that are covered in detail in Appendix C:

(1) Conversion of all elevations to NAVD 88 by readjustment or releveling.

(2) Adding a datum note based on an approximate conversion (VERTCON).

a. VERTCON. VERTCON is a software program developed by NGS that converts elevation data from NGVD 29 to NAVD 88. Although the VERTCON software has been fully incorporated into the software application package CORPSCON, it will be referred to below as a separate program. VERTCON uses benchmark heights to model the shift between NGVD 29 and NAVD 88 that is applicable to a given area. In general, it is only sufficiently accurate to meet small-scale mapping requirements. VERTCON should not be used for converting benchmark elevations used for site plan design or construction applications. Users input the latitude and longitude for a point and the vertical datum shift between NGVD 29 and NAVD 88 is reported. The root-mean-square (RMS) error of NGVD

29 to NAVD 88 conversion, when compared to the stations used to create the conversion model, is ±1 cm; with an estimated maximum error of ± 2.5 cm. Depending on network design and terrain relief, larger differences (e.g., 5 to 50 cm) may occur the further a benchmark is located from the control points used to establish the model coefficients. For this reason, VERTCON should only be used for approximate conversions where these potential errors are not critical.

b. Datum note. Whenever maps, site drawings, or spatial elevation data are provided to non-USACE users, they should contain a datum note that provides, at minimum, the following information:

> The elevations shown are referenced to the *[NGVD 29] [NAVD 88] and are in *[feet] [meters]. Differences between NGVD 29 and NAVD 88 at the center of the project sheet/data set are shown on the diagram below. Datum conversion was performed using the *[program VERTCON] [direct leveling connections with published NGS benchmarks] [other]. Metric conversions are based on *[US Survey Foot = 1200/3937 meters] [International Survey Foot = 0.3048 meters].

5-26. Vertical Transition Plan from NGVD 29 to NAVD 88

a. General. A change in the vertical datum on a project will affect USACE engineering, construction, planning, and surveying activities. The cost of conversion could be substantial at the onset. There is a potential for errors in conversions inadvertently occurring. The effects of the vertical datum change can be minimized if the change is gradually applied over time; being applied to future projects and efforts, rather than concentrated on changing already published products. In order to insure an orderly and timely transition to NAVD 88 is achieved for the appropriate products, the following general guidelines should be followed.

b. Conversion criteria. Maps, engineering site drawings, documents, and associated spatial data products containing elevation data may require conversion to NAVD 88. Specific requirements for conversion will, in large part, be based on local usage--e.g., that of the local sponsor, installation, etc. Where applicable and appropriate, this conversion should be recommended to local interests.

c. Newly authorized construction projects. Generally, initial surveys of newly authorized projects should be referenced to NAVD 88. In addition to design/construction, this would include wide-area master plan mapping work. The project control should be referenced to NAVD 88 using conventional or GPS surveying techniques. All planning and design activities should be based upon NAVD 88. All maps and site drawings shall contain datum notes as described below. If the sponsor/installation requires the use of NGVD 29 or some other local vertical reference datum for continuity, the relationship between NGVD 29 and NAVD 88 shall be clearly noted on all maps, engineering site drawings, documents, and associated products.

d. Active projects. On active projects where maps, site drawings, or elevation data are provided to non-USACE users, the conversion to NAVD 88 should be performed. This conversion to NAVD 88 may be performed the next time the project is surveyed or when the maps/site drawings are updated for other reasons. Civil works projects may be converted to NAVD 88 during the next maintenance or repair cycle in the same manner as for newly initiated civil works projects. However, if resources are not available for this level of effort, redraw the maps or drawings and add the necessary datum note. Plans should be made for the full conversion during a later maintenance or repair cycle when resources can be made available. Military installations should remain on NGVD 29 or the local vertical datum until a thoroughly coordinated effort can be arranged with the MACOM and installation. An entire installation's control network should be transformed simultaneously to avoid different datums on the same installation. MACOMs should be encouraged to convert to NAVD 88. However, the respective MACOMs are responsible for this decision.

e. Inactive projects. For inactive projects or active projects where maps, site drawings, or elevation data are not normally provided to non-USACE users, conversion to NAVD 88 is optional.

f. Work for others. Projects for other agencies will remain on NGVD 29 or the current local vertical datum until a thoroughly coordinated effort can be arranged with the sponsoring agency. Other agencies should be encouraged to convert their projects to NAVD 88, although the decision to convert rests with the sponsoring agency. However, surveys, maps, and drawings should have the datum note described below added before distribution to non-USACE users. If sponsoring agencies do not indicate a preference for new projects, NAVD 88 should be used.

g. Miscellaneous projects. Other projects referenced to strictly local datum, such as, beach nourishment, submerged offshore disposal areas, historical preservation projects, etc., need not necessarily be converted to NAVD 88. However, it is recommended that surveys, maps and drawings have a clear datum reference note added before distribution to non-USACE users.

h. Real Estate. Surveys, maps, and plats prepared in support of civil works and military real estate activities should conform as much as possible to state requirements. Many states are expected to adopt NAVD 88 (by statute) as an official vertical reference datum. This likewise will entail a transition to NAVD 88 in those states. State and local authorities should therefore be contacted to ascertain their current policies. Note that several states have adopted the International Foot for their standard conversion from meters to feet. In order to avoid dual elevations on USACE survey control points that have multiple uses, it is recommended that published elevations be based on the US Survey Foot. In states where the International Foot is the only accepted standard for boundary and property surveys, conversion of these elevations to NAVD 88 should be based on the International Foot while the control remains based on the US Survey Foot.

5-27. Vertical Control in Areas Subjected to Subsidence or Sea Level Rise

Published elevations relative to the vertical datums in high subsidence areas must be used with caution. This is due to the uneven temporal and spatial movement of the land. Thus, any geodetic or terrestrial-based elevation is not constant and must be periodically observed and adjusted for local subsidence. Likewise, hydraulic or sea level based reference datums are subject to variations due to subsidence and sea level rise at each gage site. Sea level datums also have time varying astronomical components making their reference definition more complex than terrestrial based datums. Hydraulic low water reference datums used to define navigation and flood protection elevations on the Mississippi River may also be subject to subsidence and other long-term variations, and thus these datums are spatially and temporally variable.

Subsidence is the lowering or sinking of Earth's surface, often quantified relative to non-sinking portions of the Earth's crust. It is especially pronounced in portions of California, Texas, and Louisiana. In Southern Louisiana, subsidence is occurring at a rate of up to 0.1 foot every three years in some areas. There are many potential factors that contribute to subsidence, such as the geologic composition of the area and withdrawal of ground water and oil. The rate of subsidence is not always constant and can vary from epoch to epoch (survey to survey) due to many factors, such as compaction, removal of subsurface fluids, and geologic events. Therefore, one cannot predict future subsidence with any degree of accuracy. Table 1 below illustrates the large variability in subsidence rates of change occurring over a relatively small region in Louisiana. These rates were determined from periodic First-Order, Class II leveling surveys by NOAA.

Table 1 Apparent Benchmark Movement Rates	
Benchmark Designation	**Rates of Movement in Millimeters per year**
A 148 (AU0429)	-11.01
PIKE RESET (BH1164)	-6.99
231 LAGS (BH1073)	-16.08
A 92 (BH1136)	-7.39

Subsidence can be measured and/or periodically monitored using either conventional leveling procedures or GPS techniques contained in NOAA 1997 (Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm)). Determining subsidence rates requires long-term observations and considerable analysis. As an example, Figure 5-14 below contains a map showing the estimated amount of subsidence along the route between New Orleans and Venice, LA. The leveling for this line was performed by NOAA in 1984 and adjusted to the NGVD 29 datum at that time. In 1991, NGS adjusted the entire CONUS to the NGVD 29 datum in preparation for the NAVD 88 adjustment. In Southern Louisiana, an extensive "GPS Derived Height" network was completed establishing new heights (elevations) for 85 benchmarks in Southern Louisiana. This adjustment, known as NAVD 88 (2004.65), held control outside of the subsidence area to establish new NAVD88 adjusted heights for the 85 benchmarks. Because the 1991 NAVD 88 adjustment held control outside of the area, as did the NAVD 88 (2004.65) adjustment, the change in the heights reflects the apparent movement of the marks between the observation periods. In order to determine the amount of subsidence from the time the original leveling was done, it is necessary to determine the amount of movement between the original adjustment and the 1991 national readjustment of the NGVD 29 and then the amount of movement between the original NAVD 88 adjustment and the NAVD 88 (2004.65) adjustment.

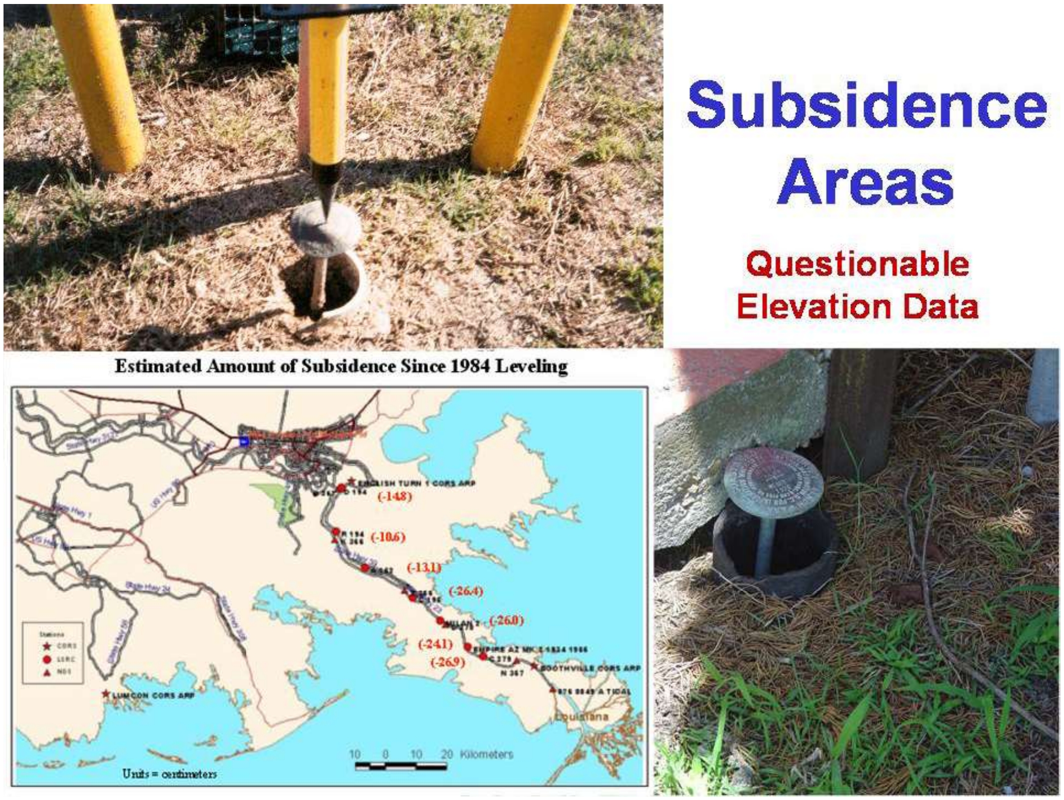

Figure 5-14. Estimated subsidence (in centimeters) in the New Orleans to Venice region. Photos depict relative ground subsidence of benchmarks attached to deep driven rods.

Monitoring subsidence or sea level changes on flood control, hurricane protection, or coastal (tidal) navigation projects requires continuous leveling or GPS surveys between water level recording gages and fixed benchmarks. Vertical datums (NAVD 88 and tidal lower low water datums) must be periodically updated to reflect changes due to subsidence or sea level rise.

5-28. Mandatory Standards

Spatial data collected for projects shall be referenced to the updated NAD 83 and NAVD 88 reference datums established by the National Oceanic and Atmospheric Administration (NOAA). Navigation projects referenced to tidal datums shall be updated to the latest tidal epoch (currently 1983-2001) in accordance with the statutory requirements in Section 224 of the Water Resources Development Act of 1992 (see Appendix B). Flood control or hurricane protection structure elevations shall be referred to the hydrodynamic surface model datum used in the design analysis.

Appendix L
Glossary

L-1. Abbreviations and Acronyms

1D .... One Dimensional
2D .... Two-dimensional
2DRMS .... Twice the distance root mean square
3D .... Three-dimensional
A-E .... Architect-Engineer
A/E/C........................... Architect/Engineer/Construction
ACSM.......................... American Congress on Surveying and Mapping
ADA .... Air Defense Artillery
AFB .... Air Force Base
ALTA .... American Land Title Association
AM/FM........................ Automated Mapping/ Facility Mapping
AOC .... Aircraft Obstruction Surveys
ARP .... Antenna Reference Point
ASCE........................... American Society of Civil Engineers
ASPRS......................... American Society for Photogrammetry and Remote Sensing
BFE.............................. Base Flood Elevation
BLM .... Bureau of Land Management
BS .... Backsight
CADD.......................... Computer Aided Drafting and Design
CAiCE .... Computer Aided Civil Engineering
CALTRANS................ California Department of Transportation
CEFMS........................ Corps of Engineers Financial Management System
COGO.......................... Coordinate Geometry
CONUS. .... CONtinental United States
CORPSCON................ CORPS CONvert
CORS .... Continuously Operating Reference Stations
COR............................. Contracting Officer's Representative
DA .... Department of the Army
DE................................ Difference in Elevation
DEM .... Digital Elevation Model
DOD .... Department of Defense
DOT............................. Department of Transportation
DFARS. .... Defense Federal Acquisition Regulation Supplement
DGPS........................... Differential Global Positioning System
DTM .... Digital Terrain Model
EAC............................. Echelons Above Corps
EDM .... Electronic Distance Measurement
EFARS......................... Engineer Federal Acquisition Regulation Supplement
EM............................... Engineer Manual
ERM .... Elevation Reference Mark
ERDC .... Engineer Research and Development Center
E&D. .... Engineering and Design
FA................................ Field Artillery
FAA............................. Federal Aviation Administration
FAC .... Florida Administrative Code

FAR .... Federal Acquisition Regulations
FAR .... Federal Aviation Regulation
FEMA .... Federal Emergency Management Agency
FFP .... Firm Fixed Price
FGCS .... Federal Geodetic Control Subcommittee
FGDC .... Federal Geographic Data Committee
FIRM .... Flood Insurance Rate Map
FIS .... Flood Insurance Study
FLSA .... Fair Labor Standards Act
FM .... Field Manual
FMSFIE .... Facility Management Standard for Facilities, Infrastructure, and Environment
FOA .... Field Operating Activity
FS................................. Foresight
G&A .... General and Administrative
GDOP .... Geometric Dilution of Position
GIS................................ Geographic Information System
GPS............................... Global Positioning System
GRS 80 .... Geodetic Reference System of 1980
GS .... General Support
GSA .... General Services Administration
GZD.............................. Grid Zone Designator
HARN .... High Accuracy Regional Networks
HI.................................. Height of Instrument
HDOP .... Horizontal Dilution of Position
HPGN .... High Precision Geodetic Networks
HR .... Height of Reflector
HT................................. Height of Target
HTRW .... Hazardous, Toxic, or radioactive Waste
HQUSACE .... Headquarters, US Army Corps of Engineers
IDC. .... Indefinite Delivery Contract
IERS .... International Earth Rotation Service
IGE .... Independent Government Estimate
IGLD 55 .... International Great Lakes Datum of 1955
IGLD 85 .... International Great Lakes Datum of 1985
ILS .... Instrument Landing System
INT .... Intersection
ITL................................ Information Technology Lab
ITRF .... International Terrestrial Reference Frame
JTR .... Joint Travel Regulation
KO .... Contracting Officer
LCC .... Lambert Conformal Conic
LEC .... Linear Error of Closure
LIDAR.......................... LIght Detection And Ranging
LWRP........................... Low Water Reference Plane
MACOM .... Major Army Command
MDL .... MicroStation Design Language
MGRS........................... Military Grid-Reference System
MHW............................ Mean High Water
MLLW.......................... Mean Lower Low Water
MLRS .... Multiple Launch Rocket System
MLS.............................. Microwave Landing System

MSL............................. Mean Sea Level
MSL1912...................... Mean Sea Level Datum of 1912
NAD 27 .... North American Datum of 1927
NAD 83 .... North American Datum of 1983
NADCON .... North American Datum Conversion
NAS............................. National Airspace System
NAVAID .... Navigation Aid
NAVD 88 .... North American Vertical Datum 1988
NDGPS......................... Nationwide Differential GPS
NFIP .... National Flood Insurance Program
NGRS .... National Geodetic Reference System
NGS .... National Geodetic Survey
NGVD 29 .... National Geodetic Vertical Datum 1929
NMAS .... National Map Accuracy Standard
NMP .... National Mapping Program
NOAA .... National Oceanic and Atmospheric Administration
NOS .... National Ocean Service
NSRS........................... National Spatial Reference System
NSSDA........................ National Standard for Spatial Data Accuracy
NVCN.......................... National Vertical Control Network
OCONUS .... Outside the Continental United States
OHWM........................ Ordinary High Water Mark
OPUS........................... On-Line Positioning User Service
OTF .... On-the-Fly
P&S .... Plans and Specifications
PADS........................... Position and Azimuth Determination System
PBM .... Permanent Benchmark
PDOP........................... Position Dilution of Position
PDSC........................... Professional Development Support Center
PI .... Point of Intersection
PLGR........................... Precise Lightweight Geodetic Receiver
PM .... Project Manager or Management
POB .... Point of Beginning
POI .... Point on Line
POT .... Point of Tangency
PPRTK .... Post-Processed Real-Time Kinematic
ppm.............................. Parts per Million
PR&C .... Purchase Request & Commitment
PRIP .... Plant Replacement and Improvement Program
PROSPECT .... Proponent Sponsored Engineer Corps Training
PVT .... Point of Vertical Tangent
QA .... Quality Assurance
QC .... Quality Control
RFP.............................. Request for Proposal
RMS .... Root mean Square
RMSE.......................... Root Mean Square Error
RTK............................. Real Time Kinematic
SCP.............................. Survey Control Point
SDSFIE........................ Spatial Data Standard for Facilities, Infrastructure, and Environment
SDTS .... Spatial Data Transfer Standard
SI .... International System of Units

SOW .... Scope or Statement of Work
SPCE .... Survey Planning and Coordination Element
SPCS .... State Plane Coordinate System
TA................................ Target Acquisition
TBM .... Temporary Benchmark
TDS .... Tripod Data Systems
TDSE........................... Touchdown Zone Elevation
TEC .... Topographic Engineering Center
TIN .... Triangular Irregular Network
TM............................... Transverse Mercator
TGO............................. Trimble Geomatics Office
TP .... Turning Point
TSC.............................. Trimble Survey Controller
US................................ United States
USACE .... US Army Corps of Engineers
USARC........................ US Army Reserve Center
USC&GS .... US Coast & Geodetic Survey
USCG .... US Coast Guard
USFS .... US Forest Service
USGS........................... US Geological Survey
USNAVOCEANO........ US Navy Oceanographic Office
USNG .... US National Grid
UTM .... Universal Transverse Mercator
VDOP .... Vertical Dilution of Position
VERTCON .... VERTical CONversion
VLBI............................ Very-Long-Baseline-Interferometry
WAAS .... Wide Area Augmentation System
WGS 84 .... World Geodetic System of 1984
WRDA......................... Water Resources Development Act
XREF........................... External Reference
ZD................................ Zenith Distance

L-2. Terms

Absolute or Autonomous GPS
Operation with a single receiver for a desired position. This receiver may be positioned to be stationary over a point. This mode of positioning is the most common military and civil application.

Accuracy
The degree to which an estimated (mean) value is compatible with an expected value. Accuracy implies the estimated value is unbiased.

Adjustment
Adjustment is the process of estimation and minimization of deviations between measurements and a mathematical model.

Altimeter
An instrument that measures elevation differences usually based on atmospheric pressure measurements.

Altitude
The vertical angle between the horizontal plane of the observer and a directional line to the object.

Angle of Depression
A negative altitude.

Angle of Elevation
A positive altitude.

Angular Misclosure
Difference in the actual and theoretical sum of a series of angles.

Archiving
Storing of documents and information.

Astronomical Latitude
Angle between the plumb line and the plane of celestial equator. Also defined as the angle between the plane of the horizon and the axis of rotation of the earth. Astronomical latitude applies only to positions on the earth and is reckoned from the astronomic equator, north and south through 90E. Astronomical latitude is the latitude that results directly from observations of celestial bodies, uncorrected for deflection of the vertical.

Astronomical Longitude
Arbitrarily chosen angle between the plane of the celestial meridian and the plane of an initial meridian. Astronomical longitude is the longitude that results directly from observations on celestial bodies, uncorrected for deflection of the vertical.

Astronomical Triangle
A spherical triangle formed by arcs of great circles connecting the celestial pole, the zenith and a celestial body. The angles of the astronomical triangles are: at the pole, the hour angle; at the celestial body, the parallactic angle; at the zenith, the azimuth angle. The sides are: pole to zenith, the co-latitude; zenith to celestial body, the zenith distance; and celestial body to pole, the polar distance.

Atmospheric Refraction
Refraction of electromagnetic radiation through the atmosphere causing the line-of-sight to deviate from a straight path. Mainly temperature and pressure conditions determine the magnitude and direction of curvature affecting the path of light from a source. Refraction causes the ray to follow a curved path normal the surface gradient.

Azimuth
The horizontal direction of a line clockwise from a reference plane, usually the meridian. Often called forward azimuth to differentiate from back azimuth.

Azimuth Angle
The angle less than 180° between the plane of the celestial meridian and the vertical plane with the observed object, reckoned from the direction of the elevated pole. In astronomic work, the azimuth angle is the spherical angle at the zenith in the astronomical triangle, which is composed of the pole, the zenith and the star. In geodetic work, it is the horizontal angle between the celestial pole and the observed terrestrial object.

Azimuth Closure
Difference in arc-seconds of the measured or adjusted azimuth value with the true or published azimuth value.

Backsight
A sight on a previously established traverse or triangulation station and not the closing sight on the traverse. A reading on a rod held on a point whose elevation has been previously determined.

Barometric Leveling
Determining differences of elevation from measured differences of atmospheric pressure observed with a barometer. If the elevation of one station above a datum is known, the approximate elevations of other station can be determined by barometric leveling. Barometric leveling is widely used in reconnaissance and exploratory surveys.

Baseline
Resultant three-dimensional vector between any two stations with respect to a given coordinate system. The primary reference line in a construction system.

Base net
The primary baseline used for densification of survey stations to form a network.

Base Points
The beginning points for a traverse that will be used in triangulation or trilateration.

Base Control
The horizontal and vertical control points and coordinates used to establish a base network. Base control is determined by field surveys and permanently marked or monumented for further surveys.

Bearing
The direction of a line with respect to the meridian described by degrees, minutes, and seconds within a quadrant of the circle. Bearings are measured clockwise or counterclockwise from north or south, depending on the quadrant.

Benchmark
A permanent material object, natural or artificial, on a marked point of known elevation.

Best Fit
To represent a given set of points by a smooth function, curve, or surface which minimizes the deviations of the fit.

Bipod
A two-legged support structure for an instrument or survey signal at a height convenient for the observer.

Bluebook
Another term for the "FGCS Input Formats and Specifications of the National Geodetic Data Base".

Blunder
A mistake or gross error.

Bureau International de l'Heure
The Bureau was founded in 1919 and its offices since then have been at the Paris Observatory. By an action of the International Astronomical Union, the BIH ceased to exist on 1 January 1988 and a new organization, the International Earth Rotation Service (IERS) was formed to deal with determination of the Earth's rotation.

Cadastral Survey
Relates to land boundaries and subdivisions, and creates units suitable for transfer or to define the limitations of title. The term cadastral survey is now used to designate the surveys of the public lands of the US, including retracement surveys for identification and resurveys for the restoration of property lines; the term can also be applied properly to corresponding surveys outside the public lands, although such surveys are usually termed land surveys through preference.

Calibration
Determining the systematic errors in an instrument by comparing measurements with correct values. The correct value is established either by definition or by measurement with a device that has itself been calibrated or of much higher precision.

Cartesian Coordinates
A system with its origin at the center of the earth and the x and y and z axes in the plane of the equator. Typically, the x-axis passes through the meridian of Greenwich, and the z-axis coincides with the earth's axis of rotation. The three axes are mutually orthogonal and form a right-handed system.

Cartesian System
A coordinate system consisting of axes intersecting at a common point (origin). The coordinate of a point is the orthogonal distance between that point and the hyperplane determined by all axes. A Cartesian coordinate system has all the axes intersecting at right angles, and the system is called a rectangular.

Celestial Equator
A great circle on the celestial sphere with equidistant points from the celestial poles. The plane of the earth's equator, if extended, would coincide with that of the celestial equator.

Celestial pole
A reference point at the point of intersection of an indefinite extension of the earth's axis of rotation and the apparent celestial sphere.

Celestial sphere
An imaginary sphere of infinite radius with the earth as a center. It rotates from east to west on a prolongation of the earth's axis.

Central Meridian
A line of constant longitude at the center of a graticule. The central meridian is used as a base for constructing the other lines of the graticule. The meridian is used as the y-axis in computing tables for a State Plane Coordinate system. That line, on a graticule, which represents a meridian and which is an axis of symmetry.

Chain
Equal to 66 feet or 100 links. The unit of length prescribed by law for the survey of the US public lands. One acre equals 10 square chains.

Chained Traverse
Observations and measurements performed with tape.

Chaining
Measuring distances on the ground with a graduated tape or with a chain.

Chart Datum
Reference surface for soundings on a nautical chart. It is usually taken to correspond to a low water elevation, and its depression below mean sea level is represented by the symbol Z_o. Since 1989, chart datum has been implemented to mean lower low water for all marine waters of the US its territories, Commonwealth of Puerto Rico and Trust Territory of the Pacific Islands.

Chi-square Testing
Non-parametric statistical test used to classify the shape of the distribution of the data.

Chronometer
A portable timekeeper with compensated balance, capable of showing time with extreme precision and accuracy.

Circle Position
A prescribed setting (reading) of the horizontal circle of a direction theodolite, to be used for the observation on the initial station of a series of stations that are to be observed.

Circuit Closure
Difference between measured or adjusted value and the true or published value.

Clarke 1866 Ellipsoid
The reference ellipsoid used for the NAD 27 horizontal datum. It is a non-geocentric ellipsoid formerly used for mapping in North America.

Closed Traverse
Starts and ends at the same point or at stations whose positions have been determined by other surveys.

Collimation
A physical alignment of a survey target or antenna over a mark or to a reference line.

Collimation Error
The angle between the actual line of sight through an optical instrument and an alignment.

Compass Rule
The correction applied to the departure (or latitude) of any course in a traverse has the same ratio to the total misclosure in departure (or latitude) as the length of the course has to the total length of the traverse.

Confidence Level

Statistical probability (in percent) based on the standard deviation or standard error associated with the normal probability density function. The confidence level is assigned according to an expansion factor multiplied by the magnitude of one standard error. The expansion factor is based on values found in probability tables at a chosen level of significance.

Conformal
Map projection that preserves shape.

Contour
An imaginary line on the ground with all points at the same elevation above or below a specified reference surface.

Control
Data used in geodesy and cartography to determine the positions and elevations of points on the earth's surface or on a cartographic representation of that surface. A collective term for a system of marks or objects on the earth or on a map or a photograph whose positions or elevation are determined.

Control Densification
Addition of control throughout a region or network.

Control Monuments
Existing local control or benchmarks that may consist of any Federal, state, local or private agency points.

Control Point
A point with assigned coordinates is sometimes used as a synonym for control station. However, a control point need not be realized by a marker on the ground.

Control Survey
A survey which provides coordinates (horizontal or vertical) of points to which supplementary surveys are adjusted.

Control Traverse
A survey traverse made to establish control.

Conventional Terrestrial Pole (CTP)
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 CTP for polar motion, as defined by the Bureau of International de l'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.

Coordinate Transformation
A mathematical process for obtaining a modified set of coordinates through some combination of rotation of coordinate axes at their point of origin, change of scale along coordinate axes, or translation through space

CORPSCON
(Corps Convert) Software package (based on NADCON) capable of performing coordinate transformations between NAD 83 and NAD 27 datums.

Crandall Method
Traverse misclosure in azimuth or angle is first distributed in equal portions to all the measured angles. The adjusted angles are then held fixed and all remaining coordinate corrections distributed among the distance measurements.

Cross sections
A survey line run perpendicular to the alignment of a project, channel or structure.

Curvature
The rate at which a curve deviates from a straight line. The parametric vector described by dt/ds, where t is the vector tangent to a curve and s is the distance along that curve.

Datum
Any numerical or geometrical quantity or set of such quantities which serve as a reference or base for other quantities.

Declination
The angle, at the center of the celestial sphere, between the plane of the celestial equator and a line from the center to the point of interest (on a celestial body).

Deflection of the Vertical
The spatial angular difference between the upward direction of a plumb line and the normal to the reference ellipsoid. Often expressed in two orthogonal components in the meridian and the prime vertical directions.

Deflection Traverse
Direction of each course measured as an angle from the direction of the preceding course.

Deformation Monitoring
Observing the movement and condition of structures by describing and modeling its change in shape.

Departure
The orthogonal projection of a line onto an east-west axis of reference. The departure of a line is the difference of the meridional distances or longitudes of the ends of the line.

Differential GPS
Process of measuring the differences in coordinates between two receiver points, each of which is simultaneously observing/measuring satellite code ranges and/or carrier phases from the NAVSTAR GPS constellation. Relative positioning with GPS can be performed by a static or kinematic modes.

Differential Leveling
The process of measuring the difference of elevation between any two points by spirit leveling.

Direction
The angle between a line or plane and an arbitrarily chosen reference line or plane. At a triangulation station, observed horizontal angles are referred to a common reference line and termed horizontal direction. A line, real or imaginary, pointing away from some specified point or locality toward another point. Direction has two meanings: that of a numerical value and that of a pointing line.

Direct Leveling
The determination of differences of elevation through a continuous series of short horizontal lines. Vertical distances from these lines to adjacent ground marks are determined by direct observations on graduated rods with a leveling instrument equipped with a spirit level.

Distance Angle
An angle in a triangle opposite a side used as a base in the solution of the triangle, or a side whose length is to be computed.

Dumpy Level
The telescope permanently attached to the leveling base, either rigidly to by a hinge that can be manipulated by a micrometer screw.

Earth-Centered Ellipsoid
Center at the Earth's center of mass and minor semi-axis coincident with the Earth's axis of rotation.

Easting
The distance eastward (positive) or westward (negative) of a point from a particular meridian taken as reference.

Eccentricity
The ratio of the distance from the center of an ellipse to its focus on the major semi-axis.

Electronic Distance Measurement (EDM)
Timing or phase comparison of electro-magnetic signal to determine an interferometric distance.

Elevation
The height of an object above some reference datum.

Ellipsoid
Formed by revolving an ellipse about its minor semi-axis. The most commonly used reference ellipsoids in North America are: Clarke 1866, Geodetic Reference System of 1980 (GRS 80), World Geodetic System of 1972 (WGS 72) and World Geodetic System of 1984 (WGS 84).

Ellipsoid height
The magnitude h of a point above or below the reference ellipsoid measured along the normal to the ellipsoid surface.

Error
The difference between the measured value of a quantity and the theoretical or defined value of that quantity.

Error Ellipse
An elliptically shaped region with dimensions corresponding to a certain probability at a given confidence level.

Error of Closure
Difference in the measured and predicted value of the circuit along the perimeter of a geometric figure.

Finite Element Method
Obtaining an approximate solution to a problem for which the governing differential equations and boundary conditions are known. The method divides the region of interest into numerous, interconnected sub-regions (finite elements) over which simple, approximating functions are used to represent the unknown quantities.

Fixed Elevation
Adopted as a result of tide observations or previous adjustment of spirit leveling, and which is held at its accepted value in any subsequent adjustment.

Foresight
An observation to the next instrument station. The reading on a rod that is held at a point whose elevation is to be determined.

Frequency
The number of complete cycles per second existing in any form of wave motion.
Geodesic Line
Shortest distance between any two points on any mathematically defined surface.

Geodesy
Determination of the time-varying size and figure of the earth by such direct measurements as triangulation, leveling and gravimetric observations.

Geodetic Control
Established and adjusted horizontal and/or vertical control in which the shape and size of the earth have been considered in position computations.

Geodetic Coordinates
Angular latitudinal and longitudinal coordinates defined with respect to a reference ellipsoid.

Geodetic Height
See Ellipsoid height.

Geodetic Latitude
The angle which the normal at a point on the reference ellipsoid makes with the plane of the equator.

Geodetic Leveling
The observation of the differences in elevation by means of a continuous series of short horizontal lines of sight.

Geodetic Longitude
The angle subtended at the pole between the plane of the geodetic meridian and the plane of a reference meridian (Greenwich).

Geodetic North
Direction tangent to a meridian pointing toward the pole defining astronomic north, also called true north.

Geodetic Reference System of 1980
Reference ellipsoid used to establish the NAD 83 system of geodetic coordinates.

Geoid
An equipotential surface of the gravity field approximating the earth's surface and corresponding with mean sea level in the oceans and its extension through the continents.

GPS (Global Positioning System)
DoD satellite constellation providing range, time, and position information through a GPS receiver system.

Gravimeter
Instrument for measuring changes in gravity between two points.

Gravity
Combined acceleration potential of an object due to gravitation and centrifugal forces.

Greenwich Meridian
The astronomic meridian through the center of the Airy transit instrument of the Greenwich Observatory, Greenwich, England. By international agreement in 1884, the Greenwich meridian was adopted as the meridian from which all longitudes, worldwide, would be calculated.

Grid Azimuth
The angle in the plane of projection between a straight line and the line (y-axis) in a plane rectangular coordinate system representing the central meridian. While essentially a map-related quantity, a grid azimuth may, by mathematical processes, be transformed into a survey- related or ground-related quantity.

Grid Inverse
The computation of length and azimuth from coordinates on a grid.

Grid Meridian
Line parallel to the line representing the central meridian or y-axis of a grid on a map. The map line parallel to the line representing the y-axis or central meridian in a rectangular coordinate system.

Gunter's Chain
A measuring device once used in land surveying. It was composed of 100 metallic links fastened together with rings. The total length of the chain is 66 feet. Also called a four-pole chain.

Gyrotheodolite
A gyroscopic device used to measure azimuth that is built-in or attached to a theodolite.

Histogram
A graphical representation of relative frequency of an outcome partitioned by class interval. The frequency of occurrence is indicated by the height of a rectangle whose base is proportional to the class interval.

Horizontal Control
Determines horizontal positions with respect to parallels and meridians or to other lines of reference.

Hour Circle
Any great circle on the celestial sphere whose plane is perpendicular to the plane of the celestial equator.

Index Error
A systematic error caused by deviation of an index mark or zero mark on an instrument having a scale or vernier, so that the instrument gives a non-zero reading when it should give a reading of zero. The distance error from the foot of a leveling rod to the nominal origin (theoretical zero) of the scale.

Indirect Leveling
The determination of differences of elevation from vertical angles and horizontal distances.

Interior Angle
An angle between adjacent sides of a closed figure and lying on the inside of the figure. The three angles within a triangle are interior angles.

International Foot
Defined by the ratio 30.48/100 meters.

International System of Units (SI)
A self-consistent system of units adopted by the general Conference on Weights and Measures in 1960 as a modification of the then-existing metric system.

Interpolation Method
Determination of a intermediate value between given values using a known or assumed rate of change of the values between the given values.

Intersection
Determining the horizontal position of a point by observations from two or more points of known position. Thus measuring directions or distances that intersect at the station being located. A station whose horizontal position is located by intersection is known as an intersection station.

Intervisibility
When two stations are visible to each other in a survey net.

Invar
An alloy of iron containing nickel, and small amounts of chromium to increase hardness, manganese to facilitate drawing, and carbon to raise the elastic limit, and having a very low coefficient of thermal expansion (about 1/25 that of steel).

Isogonic Chart
A system of isogonic lines, each for a different value of the magnetic declination.

Isogonic Line
A line drawn on a chart or map and connecting all points representing points on the earth having equal magnetic declination at a given time.

Laplace Azimuth
A geodetic azimuth derived from an astronomic azimuth by use of the Laplace equation.

Laplace Condition
Arises from the fact that a deflection of the vertical in the plane of the prime vertical will give a difference between astronomic and geodetic longitude and between astronomic and geodetic azimuth. Conversely, the observed differences between astronomic and geodetic values of the longitude and of the azimuth may both be used to determine the deflection in the plane of the prime vertical.

Laplace Equation
Expresses the relationship between astronomic and geodetic azimuths in terms of astronomic and geodetic longitudes and geodetic latitude.

Laplace Station
A triangulation or traverse station at which a Laplace azimuth is determined. At a Laplace station both astronomic longitude and astronomic azimuth are determined.

Least Count
The finest reading that can be made directly (without estimation) from a vernier or micrometer.

Least Squares Adjustment
The adjustment of the values of either the measured angles or the measured distances in a traverse using the condition that the sum of the squares of the residuals is a minimum.

Level
Any device sensitive to the direction of gravity and used to indicate directions perpendicular to that of gravity at a point.

Level Datum
A level surface to which elevations are referred. The generally adopted level datum for leveling in the US is mean sea level. For local surveys, an arbitrary level datum is often adopted and defined in terms of an assumed elevation for some physical mark.

Level Net
Lines of spirit leveling connected together to form a system of loops or circuits extending over an area.

Line of Sight
The line extending from an instrument along which distant objects are seen, when viewed with a telescope or other sighting device.

Local Coordinate System
Where the coordinate system origin is assigned arbitrary values and is within the region being surveyed and used principally for points within that region.

Local Datum
Defines a coordinate system that is used only over a region of very limited extent.

Loop Traverse
A closed traverse that starts and ends at the same station. A pattern of measurements in the field, so that the final measurement is made at the same place as the first measurement.

Magnetic Bearing
The angle with respect to magnetic north or magnetic south stated as east or west of the magnetic meridian.

Magnetic Meridian
The vertical plane through the magnetic pole including the direction, at any point, of the horizontal component of the Earth's magnetic field.

Major Semi-Axis
The line from the center of an ellipse to the extremity of the longest diameter. The term is also used to mean the length of the line.

Map
A conventional representation, usually on a plane surface and at an established scale, of the physical features (natural, artificial, or both) of a part or whole of the Earth's surface by means of signs and symbols and with the means of orientation indicated.

Map Accuracy
The accuracy with which a map represents. Three types of error commonly occur on maps: errors of representation, which occur because conventional signs must be used to represent natural or man-made features such as forests, buildings and cities; errors of identification, which occur because a non-existent feature is shown or is misidentified; and errors of position, which occur when an object is shown in the wrong position. Errors of position are commonly classified into two types: errors of horizontal location and errors of elevation. A third type, often neglected, is errors of orientation.

Map Scale
The ratio of a specified distance on a map to the corresponding distance in the mapped object.

Mean Angle
Average value of the angles.

Mean Lower Low Water (MLLW)
The average height of all lower low waters recorded over a 19-year period.
Mean Sea Level Datum
Adopted as a standard datum for heights or elevations. The Sea Level Datum of 1929, the current standard for geodetic leveling in the United States, is based on tidal observations over a number of years at various tide stations along the coasts.

Metric Unit
Belonging to or derived from the SI system of units.

Micrometer
In general, any instrument for measuring small distances very accurately. In astronomy and geodesy, a device, for attachment to a telescope or microscope, consisting of a mark moved across the field of view by a screw connected to a graduated drum and vernier. If the mark is a hair-like filament, the micrometer is called a filar micrometer.

Minor Semi-Axis
The line from the center of an ellipse to the extremity of the shortest diameter. I.e., one of the two shortest lines from the center to the ellipse. The term is also used to mean the length of the line.

Misclosure
The difference between a computed and measured value.

Monument
A physical object used as an indication of the position on the ground of a survey station.

NADCON
The National Geodetic Survey developed the conversion program NADCON (North American Datum Conversion) to convert to and from North American Datum of 1983. The technique used is based on a bi-harmonic equation classically used to model plate deflections. NADCON works exclusively in geographical coordinates (latitude/longitude).

Nadir
The point directly beneath the instrument and directly opposite to the zenith or the lowest point.

National Geodetic Vertical Datum 1929
Formerly adopted as the standard geodetic datum for heights, based on an adjustment holding 26 primary tide stations in North America fixed.

National Map Accuracy Standards
Specifications of the accuracy required of topographic maps published by the US at various scales.

National Tidal Datum Epoch
A period of 19 years adopted by the National Ocean Survey as the period over which observations of tides are to be taken and reduced to average values for tidal datums.

Network
Interconnected system of surveyed points.

Non-SI units
Units of measurement not associated with International System of Units (SI).

North American Datum of 1927
Formerly adopted as the standard geodetic datum for horizontal positioning. Based on the Clarke ellipsoid of 1866, the geodetic positions of this system are derived from a readjustment of survey observations throughout North America.

North American Datum of 1983
Adopted as the standard geodetic datum for horizontal positioning. Based on the Geodetic Reference System of 1980, the geodetic positions of this system are derived from a readjustment of survey observations throughout North America.

North American Vertical Datum of 1988
Adopted as the standard geodetic datum for heights.

Northing
A linear distance, in the coordinate system of a map grid, northwards from the east-west line through the origin (or false origin).

Open Traverse
Begins from a station of known or adopted position, but does not end upon such a station.

Optical Micrometer
Consists of a prism or lens placed in the path of light entering a telescope and rotatable, by means of a graduated linkage, about a horizontal axis perpendicular to the optical axis of the telescope axis. Also called an optical-mechanical compensator. The device is usually placed in front of the objective of a telescope, but may be placed immediately after it. The parallel-plate optical micrometer is the form usually found in leveling instruments.

Optical Plummet
A small telescope having a 90° bend in its optical axis and attached to an instrument in such a way that the line of sight proceeds horizontally from the eyepiece to a point on the vertical axis of the instrument and from that point vertically downwards. In use, the observer, looking into the plummet, brings a point on the instrument vertically above a specified point (usually a geodetic or other mark) below it.

Order of Accuracy
Defines the general accuracy of the measurements made in a survey. The order of accuracy of surveys are divided into four classes labeled: First Order, Second Order, Third Order and Fourth or lower order.

Origin
That point in a coordinate system which has defined initial coordinates and not coordinates determined by measurement. This point is usually given the coordinates (0,0) in a coordinate system in the plane and (0,0,0) in a coordinate system in space.

Orthometric Height
The elevation H of a point above or below the geoid.

Parallax
The apparent displacement of the position of a body, with respect to a reference point or system, caused by a shift in the point of observation.

Philadelphia Leveling Rod
Having a target but with graduations so styled that the rod may also be used as a self-reading leveling rod. Also called a Philadelphia rod. If a length greater than 7 feet is needed, the target is clamped at 7 feet and raised by extending the rod. When the target is used, the rod is read by vernier to 0.001 foot. When the rod is used as a self-reading leveling rod, the rod is read to 0.005 foot.

Photogrammetry
Deducing the physical dimensions of objects from measurements on photographs of the objects.

Picture Point
A terrain feature easily identified on an aerial photograph and whose horizontal or vertical position or both have been determined by survey measurements. Picture points are marked on the aerial photographs by the surveyor, and are used by the photomapper.

Planetable
A field device for plotting the lines of a survey directly from observations. It consists essentially of a drawing board mounted on a tripod, with a leveling device designed as part of the board and tripod.

Planimetric Feature
Item detailed on a planimetric map.

Plumb Line
The direction normal to the geopotential field. The continuous curve to which the gradient of gravity is everywhere tangential.

Positional Error
The amount by which the actual location of a cartographic feature fails to agree with the feature's true position.

Post-Processed Real-Time Kinematic GPS
GPS carrier phase positioning performed without real-time data link and solution.

Precision
The amount by which a measurement deviates from its mean.

Prime Meridian
The meridian of longitude 0°, used as the origin for measurement of longitude. The meridian of Greenwich, England, is almost universally used for this purpose.

Prime Vertical
The vertical circle through the east and west points of the horizon. It may be true, magnetic, compass or grid depending upon which east or west points are involved.

Project Control
Control used for a specific project.

Project Datum
Datum used for a specific project.

Projection
A set of functions, or the corresponding geometric constructions, relating points on one surface to points on another surface. A projection requires every point on the first surface to correspond one-to-one to points on the second surface.

Quadrangle
Consisting of four specified points and the lines or line segments on which they lie. The quadrangle and the quadrilateral differ in that the quadrangle is defined by four specified angle points, the quadrilateral by four specified lines or line-segments.
Random Error
Randomly distributed deviations from the mean value.

Range Pole
A simple rod fitted with a sharp-pointed, shoe of steel and usually painted alternately in red and white bands at 1-foot intervals.

Readings
The observed value obtained by noting and/or recording scales.

Real-time
An event or measurement reported or recorded at the same time as the event is occurring through the absence of delay in getting, sending and receiving data.

Real-Time Kinematic GPS
GPS carrier phase processing and positioning in real-time.

Reciprocal Leveling
Measuring vertical angles or making rod readings from two instrument positions for the purpose of compensating for the effects of refraction.

Rectangular Coordinate Systems
Coordinates on any system in which the axes of reference intersect at right angles.

Redundant Measurements
Taking more measurements than are minimally required for a unique solution.

Reference Meridian, True
Based on the astronomical meridian.

Reference Meridian, Magnetic
Based on the magnetic pole.

Reference Point
Used as an origin from which measurements are taken or to which measurements are referred.

Refraction
The bending of rays by the substance through which the rays pass. The amount and direction of bending are determined by its refractive index.

Relative Accuracy
Indicated by the dimensions of the relative confidence ellipse between two points. A quantity expressing the effect of random errors on the location of one point or feature with respect to another.

Repeating Theodolite
Designed so that the sum of successive measurements of an angle can be read directly on the graduated horizontal circle.

Resection
Determining the location of a point by extending lines of known direction to two other known points.

Sexagesimal System
Notation by increments of 60. As the division of the circle into 360°, each degree into 60 minutes, and each minute into 60 seconds.

Set-up
In general, the situation in which a surveying instrument is in position at a point from which observations are made.

Spheroid
Used as a synonym for ellipsoid.

Spirit Level
A closed glass tube (vial) of circular cross section. Its center line forms a circular arc with precise form and filled with ether or liquid of low viscosity, with enough free space left for a bubble of air or gas.

Stadia Constant
The sum of the focal length of a telescope and the distance from the vertical axis of the instrument on which the telescope is mounted to the center of the objective lens-system.

Stadia Traverse
Distances are determined using a stadia rod. A stadia traverse is suited to regions of moderate relief with an adequate network of roads. If done carefully, such a traverse can establish elevations accurate enough for compiling maps with any contour interval now standard.

Standard Error
The standard deviation of the errors associated with physical measurements of an unknown quantity, or statistical estimates of an unknown quantity or of a random variable.

Systematic Error
Errors that affect the position (bias) of the mean. Systematic errors are due to unmodeled affects on the measurements that have a constant or systematic value.

State Plane Coordinate System (SPCS)
A planar reference coordinate system used in the United States.

Strength of Figure
A number relating the precision in positioning with the geometry with which measurements are made.

Subtense Bar
A bar with two marks at a fixed, known distance apart used for determining the horizontal distance from an observer by means of the measuring the angle subtended at the observer between the marks.

Taping
Measuring a distance on the using a surveyor's tape.
Three-wire Leveling
The scale on the leveling rod is read at each of the three lines and the average is used for the final result.

Topographic Map
A map showing the horizontal and vertical locations of the natural and man-made features represented and the projected elevations of the surroundings.

Transformation
Converting a position from one coordinate system to another.

Transit
The apparent passage of a star or other celestial body across a defined line of the celestial sphere.

Transit Rule
The correction to be applied to the departure (or latitude) of any course has the same ratio to the total misclosure in departure (or latitude) as the departure (latitude) of the course has to the arithmetical sum of all the departures (latitudes) in the traverse. The transit rule is often used when it is believed that the misclosure is caused less by errors in the measured angles than by errors in the measured distances.

Transverse Mercator Projection
Mercator map projection calculated for a cylinder with axis in the equatorial plane.

Traverse
A sequence of points along which surveying measurements are made.

Triangulation
Determination of positions in a network by the measurement of angles between stations.

tribrach
The three-armed base, of a surveying instrument, in which the foot screws used in leveling the instrument are placed at the ends of the arms. Also called a leveling base or leveling head.

Trigonometric heighting
The trigonometric determination of differences of elevation from observed vertical angles and measured distances.

Trilateration
Determination of positions in a network by the measurement of distances between stations using the intersection of two or more distances to a point.

Universal Transverse Mercator
A worldwide metric military coordinate system.

US Coast & Geodetic Survey (USC&GS)
Now known as National Ocean Service (NOS).

US Survey Foot
The unit of length defined by 1200/3937 m

Variance-Covariance Matrix
A matrix whose elements along the main diagonal are called the variances of the corresponding variables; the elements off the main diagonal are called the covariances.

Vernier
An auxiliary scale used in reading a primary scale. The total length of a given number of divisions on a vernier is equal to the total length of one more or one less than the same number of divisions on the primary scaled.

VERTCON
Acronym for vertical datum conversion. VERTCON is the computer software that converts orthometric heights between NGVD 29 to NAVD 88.

Vertical Angle
An angle in a vertical plane either in elevation or depression from the horizontal.

Vertical Circle
A graduated scale mounted on an instrument used to measure vertical angles.

Vertical Datum
Any level surface used as a reference for elevations. Although a level surface is not a plane, the vertical datum is frequently referred to as the datum plane.

World Geodetic System of 1984
Adopted as the standard geodetic datum for GPS positioning. Based on the World Geodetic System reference ellipsoid.

Wye Level
Having the telescope and attached spirit level supported in wyes (Y's) in which it can be rotated about its longitudinal axis (collimation axis) and from which it can be lifted and reversed, end for end. Also called a Y-level and wye-type leveling instrument.

Zenith
The point above the instrument where an extension of a plumb (vertical) line at the observer's position intersects the celestial sphere.

Zenith Angle
Measured in a positive direction downwards from the observer's zenith to the observed target.

Zenith Distance
The complement of the altitude, the angular distance from the zenith of the celestial body measured along a vertical circle.