A. AERIAL MAPPING AND SURVEYING

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A.1. **INTRODUCTION TO GEOSPATIAL DATA**

Spatial data deals with location, shapes, and the relationships among features (topology). Site civil drawings, survey drawings, and architectural drawings are examples of spatial data normally compiled with computer-aided design and drafting (CADD) technology. Accuracies are typically depicted in relative terms—for example, boundary surveys relative to survey corners for which the geographic coordinates may be unknown in an absolute sense. Pairs of *x-y* coordinates may be referenced to an arbitrary origin, and accuracies are typically relative—for example, estimated as *n* parts per million of the distance surveyed from one point to the next. The curvature of the earth is often a negligible factor, and the rules of plane geometry typically apply.

Geospatial data refers to spatial data for which geographic coordinates are known in an absolute sense, that is, the spatial dataset is *georeferenced* to true ground coordinates. The curvature of the earth is important, and the rules of spherical geometry typically apply. Positioning is relative to geodetic data, using control surveys having geodetic *network accuracy*, rather than *local accuracy* relative to an arbitrary origin. Geospatial data is georeferenced as 2-D or 3-D coordinates of points on, above, or below a mathematical model of the earth (ellipsoid). Horizontal positions may be expressed in terms of *geographic coordinates,* that is, longitude (λ) east or west of the Greenwich Meridian and latitude (ϕ) north or south of the equator. For land development general planning purposes, horizontal positions are normally expressed as 2-D rectangular coordinate pairs, that is, easting (*x*) and northing (*y*) coordinates relative to a *horizontal datum* and coordinate system origin (where *x* and *y* coordinates are zero) normally defined by the State Plane Coordinate System (SPCS). For detailed planning purposes, a *z* value is added to each *x-y* coordinate pair in order to define elevations relative to a *vertical datum* and origin where the elevation (*z* value) is zero. These 3-D coordinates may be obtained from ground surveying (as described in detail in other chapters) or aerial surveying and mapping as described in this appendix, to include photogrammetry and light detection and ranging (LiDAR). These 3-D coordinates could also be obtained from various forms of geographic information systems (GIS).

In some states, those engaged in aerial mapping and surveying require professional licensure (e.g., photogrammetric surveyors), much like professional land surveyors and professional engineers. The reason for this is that state definitions for the “practice of land surveying” require licensure for those who determine authoritative positions, coordinates, and topography for which an unqualified practitioner could adversely impact public safety and welfare, regardless of whether such data are produced by land surveyors or aerial surveyors. A few states even require those producing GIS datasets to be licensed, even though they generally use, overlay, and/or reformat data produced by others rather than provide authoritative positioning.

A.1.1. **Types of Geospatial Data**

This section is important because failure to understand the different types of geospatial data could cause the wrong data to be requested, resulting in subcontracts for data unsuitable for its intended purpose and/or requiring additional expenses to correct misunderstandings.

Raster Data and Vector Data. *Raster data* consists of pixels or grid cells of uniform resolution. Digital images are raster data, as are maps or engineer drawings scanned at

500 or 1000 dots per inch (dpi), for example. A *pixel* (picture element) is the smallest indivisible element of a digital image. One pixel of a LANDSAT satellite multispectral image equates to a 30-meter by 30-meter square area on the ground; 1 pixel of a standard USGS Digital Orthophoto Quarter-Quad (DOQQ) covers a 1-meter by 1-meter square area, and 1 pixel of a high-resolution digital orthophoto typically produced specifically for a land development project might cover a 6-inch by 6-inch area on the ground. Such images are referred to as having 30-meter, 1-meter, or 6-inch pixel resolution, respectively. However, users can typically see features that are much smaller than the pixel resolution; for example, road paint stripes, 4 inches wide, can often be seen on DOQQs with 1-meter pixel resolution. The resolution of scanned documents is referred to in terms of dpi. Raster images can be displayed on the computer at different scales, to the point where *pixel breakdown* occurs and individual pixels are visible, but zooming in does not improve the accuracy of such raster data.

Somewhat different from a pixel, a raster *grid cell* is one element of a more detailed image or surface, simplified for cost and convenience using user-defined grid-spacing criteria. Square grids are commonly defined to reduce the computer file storage requirements for large geospatial datasets. For example, whereas it might be desirable to utilize digital orthophotos with 6-inch pixel resolution as base maps, larger grid cells of 1 meter, 10 meters, 50 meters, and larger are often preferred to display soil types, geology, vegetation classification, land use/land cover, and natural features that do not need to be precisely depicted. Similarly with elevation data, whereas it might be necessary to collect randomly spaced LiDAR data with average point density of 4 points per square meter in order to maximize the potential for penetrating vegetation to map the bare-earth terrain beneath the trees, the file sizes are very large for recording *x-y-z* coordinates for millions/billions of points, and the analysis/display software is more expensive. File sizes are much smaller when using a digital elevation model (DEM) grid spacing of 1 meter (are larger) for which *x-y* coordinates do not need to be individually stored and only the average *z* value is stored for each square grid cell. Instead of complex horizontal coordinates, grid cells are tracked by sequential rows and columns.

Vector data consists of 2-D (*x-y*) or 3-D (*x-y-z*) coordinates defining the locations of point, line, and area features. Related terms include *nodes, vertices, shape points, arcs, degenerate lines, line strings, line chains, edges, polygons,* and other terms that have their own definitions. Vector data may be displayed on a computer with different colors, line styles (solid, dashed, dotted, etc.), line weights (thicknesses), and symbols. Vector lines and curves are normally smooth, while grid cell raster data have a stair-step appearance, especially when zoomed-in to view the raster data at a large scale. CADD drawings are a form of vector data, as are survey data with points having 2-D or 3-D coordinates, and lines having distances and bearings.

Merged raster/vector data is now common with modern GISs. Raster images are more understandable to humans, but attributed vector data is more intelligible to computers. The merger of raster and vector data (normally the overlay of vector data on top of raster images) allows the best of both worlds. The raster data normally serves as a base map for overlay of vector GIS data.

Descriptive Data and Map Annotations. *Descriptive data* more fully describes the geospatial data. *Attributes* describe the various point, line, and area features, such as length, diameter, manufacturer, model, serial number, material composition, burial depth, and installation data of a utility feature. Often, feature attribute codes are used to describe diverse feature types. Alphanumeric textual data or attributes are often stored in relational databases, and selected items from the database may be displayed on maps. Descriptive databases can be managed separately from geospatial data; for example, a tax assessor's database may be linked to a county's parcel maps but maintained separately and confidentially from spatial databases available to the general public.

Annotations are map symbols or alphanumeric labels such as route numbers, elevations, or names of towns, streets, rivers, or mountains placed on maps. On digital maps, annotations are normally key-entered, then stored with coordinates for the beginning, center, or ending of the annotation and/or with other rules for placement and orientation, text fonts, and special characters and symbols of various sizes.

Planimetric Maps and Topographic Maps. *Planimetric maps* display the horizontal positions of natural and manmade features and boundary lines. Planimetric data is displayed in two dimensions only. If maps do not display elevation data with contour lines or an alternative method, they are planimetric maps. In a computer, planimetric data is treated as 2-D files having *x-y* coordinates. Digital orthophotos are a form of planimetric map where no elevation data is presented. Planimetric maps and planimetric data normally form the base maps for overlay of other data required for a land development project.

When people refer to *planimetrics,* they normally mean planimetric data that can be seen and mapped horizontally, typically from stereo photogrammetry described later. Planimetrics include hydrographic features (e.g., rivers, lakes, and shorelines), transportation features (e.g., road/highway edge of pavement, bridges, railroad tracks, and airport runways and taxiways), man-made features (e.g., building footprints, transmission lines, fire hydrants), and other features that can be seen from aerial photography. The term *planimetrics* does not normally include boundary lines and underground utilities that cannot be mapped with stereo aerial photography, even though they can be surveyed and georeferenced on planimetric maps. Similarly, a *plan view* shows the horizontal location of features as though looking straight down from infinity.

Topographic maps display both the horizontal positions of natural and man-made features and boundary lines as well as elevation data (normally contour lines). In a computer, topographic data may be 3-D files having *x-y-z* coordinates for individual points or line strings, or they may be different forms of 2-D files with at least one file including contour lines with elevation attributes. Topographic data and topographic surveys normally end at the edge of water bodies, but they often include the elevation of a lake.

Both planimetric data and topographic data are produced by field surveying, photogrammetric mapping, and/or remote sensing. Topographic data is also produced by new remote sensing technologies, especially LiDAR, for which the intensity images approximate an orthophoto image and for which lidargrammetry can be used to generate 2-D or 3-D breaklines. Whether produced from photogrammetry or lidargrammetry, breaklines are linear features that describe a change in the smoothness or continuity of a surface. The two most common forms of breaklines are as follows:

■ A *soft breakline* ensures that known *z* values along a linear feature are maintained (e.g., elevations along a pipeline, road centerline, or drainage ditch) and that linear features and polygon edges are maintained in a triangulated irregular network (TIN) surface model, by enforcing the breaklines as TIN edges. They are generally synonymous with 3-D breaklines because they are depicted with series of *x-y-z* coordinates. Somewhat rounded ridges (road crowns) or the trough of a drain may be collected using soft breaklines.

■ A *hard breakline* defines interruptions in surface smoothness, e.g., to define streams, shorelines, dams, ridges, building footprints, and other locations with abrupt surface changes. Although some hard breaklines are 3-D breaklines, they are often 2-D breaklines because features such as shorelines and building footprints are normally depicted with series of *x-y* coordinates only, often digitized from digital orthophotos that include no elevation data.

Mass points are irregularly spaced points, each with*x-y* location coordinates and *z* value, typically (but not always) used to form a TIN. When generated manually—by a photogrammetrist, for example—mass points are ideally chosen to depict the most significant variations in the slope or aspect of TIN triangles. However, when generated automatically—for example, by photogrammetric automated image correlation, LiDAR, or IFSAR—mass point spacing and pattern depend on the characteristics of the technologies used to acquire the data and postspacing criteria selected by the operator.

Digital Elevation Models (DEMs). DEMs have at least three different meanings to different users. For some, DEM is a generic term for digital topographic and/or bathymetric data in all its various forms. For the U.S. Geological Survey (USGS), a DEM is a standard form of elevation dataset at regularly spaced intervals in *x* and *y* directions georeferenced in Universal Transverse Mercator (UTM) coordinates (with uniform 30-meter or 10-meter grid spacing) or geographic coordinates (with uniform 1-arc-second or 1/3-arc-second grid spacing); some data is now available in 1/9-arc-second, approximately 3-meter spacing at the equator. For others in the United States, a DEM has *z* values at regularly spaced intervals in*x* and *y* directions, but with alternative specifications, such as narrower grid spacing and State Plane coordinates. DEMs always imply elevation of the terrain (bare-earth *z* values) devoid of vegetation and man-made features, as opposed to digital surface models (DSMs) that include the elevations of treetops, rooftops, towers, and other features raised above the terrain. In Europe, DEMs are considered to be synonymous with digital terrain models (DTMs), but in the United States DTMs include irregularly spaced mass points and/or breaklines where the slope changes, thereby depicting the true shape of the terrain more accurately than a gridded DEM. For more information on this subject, see *Digital Elevation Model Technologies and Applications: The DEM Users Manual,* published in 2007 by the American Society for Photogrammetry and Remote Sensing (ASPRS). DEMs, DTMs, and DSMs are efficiently used for computer analysis and display of the topographic surface.

Contours. Contours are lines of equal elevation. They are intended exclusively for human interpretation and have little if any value for computer analyses of the terrain. Contours have traditionally been produced by stereo photogrammetric compilation, where the operator can see the breaklines (where the slope changes) and can manually shape the contour lines so that they are aesthetically pleasing. When manually compiling maps using photogrammetry, the compiler also shapes contours (according to established rules) where they cross streams and roads.

When contours are generated automatically from DEMs, they are not as aesthetically pleasing. A DEM has no way of knowing where a breakline exists between DEM points and, therefore, can't automatically shape the contours to depict streams, roads, retaining walls, and so on, correctly. Breaklines are added to correct for this limitation. It is much simpler to convert contours into DEMs (for computer analysis of the terrain) than to convert DEMs into contours (for visual analysis of the terrain).

Digital Orthophotos. A digital orthophoto has the image qualities of an aerial photograph but the metric properties of a map. A digital orthophoto is a digital image from a perspective photo or image, corrected by an orthorectification process so as to remove *tilt displacement* (caused by the roll, pitch, and yaw of the aircraft in flight) and*relief displacement* (caused by the perspective view of the aerial photograph, which causes taller objects to appear larger and closer than they really are to the camera). Digital images are produced either by acquiring the aerial images with a digital camera or by scanning aerial film. Processes for removing tilt displacement and relief displacement are explained in the section on photogrammetry later in this chapter.

A.2. **NATIONAL SPATIAL DATA INFRASTRUCTURE (NSDI)**

A.2.1. **Framework Data**

The NSDI is defined as the technologies, policies, and people necessary to promote sharing of geospatial data throughout all levels of government, the private and nonprofit sectors, and the academic community. The NSDI *framework* is a collaborative effort to create a widely available source of basic geographic data. It provides the most common data themes geographic data users need, as well as an environment to support the development and use of these data. The framework represents "data you can trust"—the best available data for an area, certified, standardized, and described according to a common standard.

The NSDI framework provides a foundation on which organizations can build by adding their own detail and compiling other datasets. Its key aspects are: (1) seven themes of digital geographic data that are commonly used, including geodetic control, orthoimagery, elevation, transportation, hydrography, governmental units, and cadastral information; (2) procedures, technology, and guidelines that provide for integration, sharing, and use of this data; and (3) institutional relationships and business practices that encourage the maintenance and use of the data.

A.2.2. **National Spatial Reference System (NSRS)**

The NSRS is a consistent coordinate system that defines latitude, longitude, height, scale, gravity, and orientation throughout the United States, and how these values change with time. The NSRS comprises the following:

1. The National CORS, a set of global positioning system (GPS) continuously operating reference stations (CORS) meeting the highest standards for defining the geodetic datum.

2. A network of permanent survey monuments including the Federal Base Network (FBN), Cooperative Base Network (CBN), and User Densification Network (UDN).

3. A consistent, accurate, and up-to-date national shoreline.

4. A set of accurate models describing dynamic geophysical processes that affect spatial measurements.

The NSRS provides a highly accurate, precise, consistent geographic framework throughout the United States. It is the foundation for the NSDI and a significant national resource. It includes hundreds of thousands of geodetic control monuments and benchmarks that have been surveyed relative to our horizontal and vertical datums (i.e., NAD 83 and NAVD 88). As per the NSDI framework motto, the NSRS represents survey control "data you can trust"—the best available survey data for any area within the United States. It's accurate, it's readily available on the Web, it's simple to use, and it's free. NSRS data should be used as the foundation for any land development survey project.

To locate the most suitable NSRS survey monuments for any project area, visit the National Geodetic Survey (NGS) home page at www.ngs.noaa.gov, then click on "Data Sheets." Under "Search the NGS DataBase and Retrieve Data Sheets," select either "radial search" or "rectangular search." Under "radial search," enter the latitude and longitude of the center of the land development project area or the estimated geographic coordinates of the local network control point to be surveyed, plus the circular search radius in miles, up to 20 miles maximum. Alternatively, under "rectangular search," enter the min-max values for the latitude and longitude of the rectangle surrounding the project site or the local network control point to be surveyed. NGS data sheets will be listed for all NSRS monuments within the search area so that the best monuments can be selected for extension of subsequent control. If more than 100 NSRS control points exist within the specified area, the user needs to subdivide the rectangular search area or narrow the radial search area in order to fit below the 100 maximum data sheets that can be printed from the website.

Last, the user will be asked to select survey control points in one of three categories: (1) any horizontal and/or vertical control point (surveyed by any method—conventional or GPS), (2) any horizontal control points only (surveyed by any method—conventional or GPS), and (3) GPS sites only.

A.3. **FEDERAL GEOGRAPHIC DATA COMMITTEE (FGDC) ACCURACY STANDARDS**

For a half century, nearly all maps were produced in accordance with the National Map Accuracy Standard (NMAS) published in 1947. But the NMAS was replaced in 1998 by the FGDC Geospatial Positioning Accuracy Standards for digital geospatial data. It is important for land development surveyors and engineers to understand the differences between the new and the old standards. This subsection first summarizes the old standards before comparing them with the new standards.

A.3.1. **National Map Accuracy Standard (NMAS)**

According to the NMAS (Bureau of the Budget, 1947), horizontal and vertical accuracy of maps are defined as follows:

■ For maps produced at a scale of 1:20,000 and larger (which includes most land development projects), “not more than 10 percent of the points tested shall be in error by more than 1/30 inch, measured on the publication scale.... These limits of accuracy shall apply in all cases to positions of well defined points only. ‘Well defined’ points are those that are easily visible or recoverable on the ground, such as the following: monuments or markers, such as bench marks, property boundary monuments, intersections of roads, railroads, etc.; corners of large buildings or structures (or center points of small buildings), etc. In general what is ‘well defined’ will also be determined by what is plottable on the scale of the map within 1/100 inch.” At map scale, the Circular Map

Accuracy Standard (CMAS) equals 1/30th of an inch; at ground scale, the CMAS equals 1/30th of an inch divided by the map scale. For example, DOQQs compiled at 1 in = 1,000 ft have a CMAS of 1/30th of an inch on the map or 33.3 ft on the ground.”

■ “Vertical accuracy, as applied to contour maps on all publication scales, shall be such that not more than 10 percent of the elevations tested shall be in error more than one-half the contour interval. In checking elevations taken from the map, the apparent vertical error may be decreased by assuming a horizontal displacement within the permissible horizontal error for a map of that scale.” For example, the Vertical Map Accuracy Standard (VMAS) equals one-half the contour interval on the map, or 1.0 foot in ground elevation if the contour interval is 2 feet.

Both the CMAS and the VMAS are standards at the 90 percent confidence level, but these NMAS errors do not necessarily follow a normal distribution—that is, there are no limits to the size of the errors that exceed the CMAS or VMAS. The NMAS is now obsolete for use with digital geospatial data.

A.3.2. **Geospatial Positioning Accuracy Standards (GPAS)**

With digital data, the geospatial data community recognized that the NMAS standards were inappropriate because the scale and contour interval of a digital map could be changed with the click of a computer mouse, without changing the underlying accuracy of the data. Just because one can easily zoom in on GIS data and view the data at high resolution does not mean that the data suddenly gets more accurate.

In 1990, the American Society for Photogrammetry and Remote Sensing (ASPRS) published the ASPRS standards for Large Scale Maps (ASPRS, 1990). These ASPRS 90 standards were modified subsequently by the Federal Geographic Data Committee (FGDC) in the FGDC Geospatial Positioning Accuracy Standards (GPAS), published in 1998, which officially replaced the NMAS for digital geospatial data. Although the GPAS has three parts, almost everyone refers to Part 3—the NSSDA—when referring to the GPAS.

■ Part 1: Reporting Methodology (FGDC, 1998a)

■ Part 2: Standards for Geodetic Networks (FGDC, 1998b)

■ Part 3: National Standard for Spatial Data Accuracy (NSSDA)(FGDC, 1998c)

Part 1: Reporting Methodology. Reference FGDC-STD007.1-1998 at www.fgdc.gov/standards/documents/standards/chapter1.pdf.

Part 1 provides a common methodology for reporting the accuracy of horizontal and/or vertical coordinate values for clearly defined features where the location is represented by a single-point coordinate, either 2-D or 3-D. Examples are survey monuments; prominent landmarks such as church spires, standpipes, radio towers, tall chimneys, and mountain peaks; and targeted photogrammetric control points. It provides the means to compare directly the accuracy of coordinate values obtained by one method (e.g., a cartographically derived value) with that obtained by another method (e.g., a GPS geodetic network survey) for the same point. This helps the users to know not only the coordinate values but also the accuracy of those coordinate values, so they can decide which coordinate values represent the best estimate of the true value for their applications.

According to FGDC, 1998a, horizontal and vertical accuracy are to be reported in ground distances at the 95 percent confidence level, as follows:

■ Horizontal: "The reporting standard in the horizontal component is the radius of a circle of uncertainty, such that the true or theoretical location of the point falls within that circle 95-percent of the time." The National Standard for Spatial Data Accuracy (NSSDA) refers to this horizontal accuracy component as "Accuracy $_r$."

■ Vertical: "The reporting standard in the vertical component is a linear uncertainty value, such that the true or theoretical location of the point falls within ± of that linear uncertainty value 95-percent of the time." The NSSDA refers to this vertical accuracy component as "Accuracy $_z$."

Furthermore, accuracy is to be reported in terms of local accuracy or network accuracy.

■ "The *local accuracy* of a control point is a value that represents the uncertainty in the coordinates of the control point relative to the coordinates of other directly connected, adjacent control points at the 95-percent confidence level. The reported local accuracy is an approximate average of the individual accuracy values between this control point and other observed control points used to establish the coordinates of the control point." (For practical purposes, *local accuracy* can be considered the equivalent of *relative accuracy,* defined in other FGDC standards as "point-to-point accuracy.")

■ "The *network accuracy* of a control point is a value that represents the uncertainty in the coordinates of the control point with respect to the geodetic datum at the 95-percent confidence level. For NSRS network accuracy classification, the datum is considered to be best expressed by the geodetic values at the Continuously Operating Reference Stations (CORS) supported by NGS. By this definition, the local and network accuracy values at CORS sites are considered to be infinitesimal, i.e., to approach zero." (For practical purposes, *network accuracy* can be considered the equivalent of *absolute accuracy,* defined in other FGDC standards as "accuracy stated with respect to a defined datum or reference system.")

Part 2: Standards for Geodetic Networks. Reference FGDC-STD-007.2-1998 at www.fgdc.gov/standards/documents/standards/chapter2.pdf.

Part 2 (FGDC, 1998b) provides a common methodology for determining and reporting the accuracy of horizontal and vertical coordinate values for geodetic control points represented by survey monuments such as brass disks and rod marks. It provides a means to compare directly the accuracy of coordinate values obtained by one method (e.g., a classical line-of-sight traverse) with the accuracy of coordinate values obtained by another method (e.g., a GPS geodetic network survey) for the same point.

According to FGDC, 1998b, geodetic control surveys are usually performed to establish a basic control network (framework) from which supplemental surveying and mapping work is performed (e.g., for land development). Geodetic network surveys are distinguished by use of redundant, interconnected, permanently monumented control points that constitute the framework for the NSRS or are often incorporated into the NSRS. Such surveys must be performed to far more rigorous accuracy and quality assurance standards than those for control surveys for general engineering or construction. However, geodetic control surveys are required for controlling interstate transportation corridors (e.g., highways, pipelines, and railroads), long-span bridge construction alignment, geophysical studies, and structural deformation monitoring of dams, buildings, and similar facilities.

The NSRS supports both local accuracy and network accuracy. Local accuracy is best adapted to check relations between nearby control points; for example, a surveyor checking closure between two NSRS points is mostly interested in a local accuracy measure. On the other hand, someone constructing a GIS or LIS often needs some type of positional tolerance associated with a set of coordinates. Network accuracy measures how well coordinates approach the error-free datum. To establish the network accuracy of a control point, it is not necessary to connect directly to a CORS; however, it is necessary that the survey be connected properly to existing NSRS control points with established network accuracy values.

Part 3: National Standard for Spatial Data Accuracy (NSSDA). Reference FGDC-STD-007.3-1998 at www.fgdc.gov/standards/documents/standards/chapter3.pdf.

The NSSDA (FGDC, 1998c) implements a statistical and testing methodology for estimating the positional accuracy of points on maps and in digital geospatial data, with respect to georeferenced ground positions of higher accuracy. The NSSDA applies to fully georeferenced maps and digital geospatial data, in either raster, point, or vector format, derived from sources such as aerial photographs, satellite imagery, and ground surveys. Contrary to the National Map Accuracy Standard, which is now obsolete, the NSSDA does not define threshold accuracy values. Producing agencies and users must identify acceptable accuracies for their applications. Data and map producers must determine what accuracy exists or is achievable for their data and report it according to the NSSDA.

According to FGDC, 1998c, “Accuracy is reported in ground distances at the 95% confidence level. Accuracy reported at the 95% confidence level means that 95% of the positions in the dataset will have an error with respect to true ground position that is equal to or smaller than the reported accuracy value. The reported accuracy value reflects all uncertainties, including those introduced by geodetic control coordinates, compilation, and final computation of ground coordinate values in the product.”

The NSSDA uses root-mean-square error (RMSE) to estimate positional accuracy. RMSE is the square root of the average of the set of squared differences between dataset coordinate values and coordinate values from an independent source of high accuracy for identical points.

Well-established procedures (e.g., field surveys or photogrammetry with fully analytical aerial triangulation) report accuracy as follows: “Compiled to meet ____ (meters, feet) horizontal accuracy at 95% confidence level; compiled to meet ____ (meters, feet) vertical accuracy at 95% confidence level.”

New or questionable procedures (e.g., LiDAR or photogrammetry without fully analytical aerial triangulation) should be tested with accuracy reported as follows: “Tested (meters, feet) horizontal accuracy at 95% confidence level; tested ____ (meters, feet) vertical accuracy at 95% confidence level.”

When determined to be required, accuracy testing by an independent source of higher accuracy is the preferred test for positional accuracy. Horizontal accuracy should be tested by comparing the planimetric coordinates of well-defined points in the dataset with coordinates of the same points from an independent source of higher accuracy. A well-defined point represents a feature for which the horizontal position is known to a high degree of accuracy and position with respect to the geodetic datum; for the purpose of accuracy testing, well-defined points must be easily visible or recoverable on the ground, on the independent source of higher accuracy, and on the mapping or GIS product itself. Vertical accuracy should be tested by comparing the elevations in the dataset with elevations of the same points as determined from an independent source of higher accuracy; graphic-contour data and digital topographic data may not contain well-defined points; therefore, the accuracy of a DEM is computed by a comparison of linearly interpolated elevations in the DEM with corresponding known elevations of checkpoints of higher accuracy. ASPRS, 2007, provides guidance on the selection of appropriate vertical checkpoints as well as pitfalls to be avoided in vertical accuracy testing.

Table A.1 NMAS and NSSDA Horizontal Accuracy Equivalencies

MAP SCALE OR COMPILATION SCALE	NMAS, CMAS AT 90% CONFIDENCE LEVEL	NSSDA, ACCURACY $_r$ AT 95% CONFIDENCE LEVEL	RMSE $_r$
1 in = 50 ft or 1:600	1/30 in = 1.67 ft ground distance	1.90 ft ground distance	1.10 ft ground distance
1 in = 83.3 ft or 1:1000	1/30 in = 2.78 ft ground distance	3.17 ft ground distance	1.83 ft ground distance
1 in = 100 ft or 1:1200	1/30 in = 3.33 ft ground distance	3.80 ft ground distance	2.20 ft ground distance
1 in = 200 ft or 1:2400	1/30 in = 6.67 ft ground distance	7.60 ft ground distance	4.39 ft ground distance

Normally, a minimum of 20 checkpoints should be tested; then the 95 percent confidence level allows 1 point to fail the threshold given in product specifications.

For horizontal accuracy testing, the radial RMSE ($RMSE_r$) is determined; then Accuracy $_r$ = 1.7308 × RMSE $_r$, where $RMSE_r = \sqrt{RMSE_x^2 + RMSE_y^2}$.

If horizontal errors are believed to have a normal distribution, a conversion can be made between the NMAS's Circular Map Accuracy Standard (CMAS) and NSSDA's RMSE $_r$ and Accuracy $_r$ as follows:

- CMAS = 1.5175 × RMSE $_r$
- Accuracy $_r$ = 1.1406 × CMAS

Table A.1 provides examples of equivalent horizontal accuracy standards, comparing the NMAS accuracy thresholds (90 percent confidence level) for specified map scales with comparable accuracy reported per NSSDA terminology in terms of the RMSE $_r$ and Accuracy $_r$ (horizontal radial accuracy at the 95 percent confidence level).

For vertical accuracy testing, the vertical RMSE ($RMSE_z$) is determined; then Accuracy $_z$ = 1.9600 × RMSE $_z$. If elevation errors are believed to have a normal distribution, a conversion can be made between the NMAS's Vertical Map Accuracy Standard (VMAS) and NSSDA's RMSE $_z$ and Accuracy $_z$ as follows:

- VMAS = 1.6449 × RMSE $_z$
- Accuracy $_z$ = 1.1916 × VMAS
- Contour Interval = 3.2898 × RMSE $_z$

Table A.2 provides examples of equivalent vertical accuracy standards, comparing the NMAS accuracy thresholds (90 percent confidence level) for specified contour intervals with comparable accuracy reported per NSSDA terminology in terms of RMSE $_z$ and Accuracy $_z$ (vertical accuracy at the 95 percent confidence level).

It should be noted that the NMAS/NSSDA equivalencies in Tables A.1 and A.2 can be made only by assuming that all errors have a normal distribution. In fact, the NMAS never made this assumption, and 10 percent of the errors are allowed to exceed the threshold values at the 90 percent confidence level, regardless of size. The NMAS outliers could presumably be very large errors, whereas the NSSDA outliers would presumably be relatively small, or else they would skew the RMSE calculations disproportionately. Because the remote sensing community knows that vertical errors in bare-earth LiDAR datasets sometimes do not follow a normal error distribution in vegetated terrain where the vegetation filtering process does not necessarily follow a normal error distribution, alternative procedures for testing and reporting LiDAR data are documented in ASPRS, 2007, consistent with guidelines from the National Digital Elevation Program (NDEP) and ASPRS LiDAR accuracy reporting guidelines.

Table A.2 NMAS and NSSDA Vertical Accuracy Equivalencies

CONTOUR INTERVAL AT WHICH COMPILED	NMAS, VMAS AT 90% CONFIDENCE LEVEL	NSSDA, ACCURACY $_z$ AT 95% CONFIDENCE LEVEL	RMSE $_z$
6 in	3 in	3.57 in or 9.08 cm	1.82 in or 4.63 cm
1 ft	6 in	7.15 in or 18.16 cm	3.65 in or 9.27 cm
2 ft	12 in	14.30 in or 36.32 cm	7.30 in or 18.53 cm
5 ft	30 in	35.75 in or 90.80 cm	18.24 in or 46.33 cm

A.4. **AERIAL MAPPING DISCIPLINES**

CADD drawings and GIS databases, described in other chapters and appendices, are often based on source data using disciplines/technologies described here.

■ A basic understanding of geodesy is required for larger land development projects, because geodesy makes maps accurate.

■ A basic understanding of photogrammetry is required for aerial image acquisition and production of accurate base map information, including digital orthophotos and planimetrics.

■ A comparative understanding of photogrammetry and lidargrammetry is required for acquisition of accurate topographic data, including DEMs, mass points, breaklines, and contours for areas too large to be mapped efficiently using ground survey.

Land development projects, both large and small, require detailed mapping of the current topography, natural and man-made features, and overlay of other relevant geospatial data in a GIS used by engineers in making key decisions. Small mapping projects can be performed by ground surveys, including conventional surveys, GPS surveys, closerange photogrammetric surveys, and terrestrial laser scanning. Large mapping projects are performed by aerial surveys, including aerial photography from film or digital cameras, photogrammetric mapping, and LiDAR mapping of the bareearth terrain and/or top reflective surfaces (e.g., treetops, rooftops, towers); but they also include ground surveys to map features that are not visible from the air, such as survey monuments, survey corners, boundary lines, culverts, and underground utilities.

Land development routinely requires analyses of land information, flood and erosion hazards, environmental impact, and access to and impact on supporting infrastructure. Land development projects may also require analyses of transportation networks, hydrologic networks, water quality, environmental restoration, and hazardous waste management. The goal of a responsible land development project manager is to eliminate, or minimize, adverse impacts on others. To do this, various forms of mapping support are essential.

The remainder of this appendix focuses on three geospatial disciplines/technologies used in aerial mapping and surveying:

■ *Geodesy,* including ellipsoids and horizontal datums, map projections and state plane coordinate systems, heights and vertical datums, scale factors, and major points to understand about geodesy

■ *Photogrammetry,* including aerial and terrestrial photography, topographic and planimetric mapping, digital orthophotography, oblique aerial imaging, and major points to understand about photogrammetry

■ *LiDAR,* including mapping of the bare-earth topographic surface, top reflective surfaces, lidargrammetry in generation of 3-D breaklines, and major points to understand about LiDAR

A.5. **GEODESY**

“Mapping without geodesy is a felony.” This popular sign, displayed on the wall at the National Geodetic Survey, is meant with tongue in cheek; however, mapping without understanding of geodesy can get mapping firms into legal trouble. Misunderstanding the basics of geodesy has cost some firms millions of dollars to correct basic mistakes. Examinations for licensure or certification of photogrammetrists always include basic questions on geodesy.

Geodesy is the science of determining the size and shape of the earth and its gravity field, and the precise location of points on the earth's surface. Generally speaking, *geometric geodesy* is used to define horizontal positions, consistent with the rules of spherical geometry, and *physical geodesy* is used to define elevations, consistent with the rules of gravity. Whenever surveyors level their conventional survey instruments over points on the earth, they are establishing level lines or vertical angular orientations relative to their local direction of gravity—directions that vary with changing mass and density of the earth. Therefore, conventional ground surveys follow the *rules of gravity,* whereas global positioning system (GPS) surveys follow the *rules of geometry.* A surveyor or mapper needs to understand these distinctions in order to make accurate topographic maps used for land development and other applications.

Land development projects sometimes encounter difficulties when engineers or other key personnel do not understand or pay close enough attention to the following:

- Differences between horizontal datums
- Differences between geographic, state plane, and UTM coordinates
- Differences between true north, grid north, and magnetic north
- Differences between vertical datums
- Differences between ellipsoid heights, orthometric heights, and geoid heights
- Differences between measurement units, especially the U.S. survey foot and the international foot
- Formulas and significant figures necessary for accurate coordinate transformations

This section on geodesy is intended to help engineers and planners understand these differences as well as basic geodetic terms that could prevent needless problems from occurring with mapping and land development projects.

A.5.1. **Geoid**

The force acting on a body at rest on the earth's surface results from the gravitational force and the centrifugal force of the earth's rotation. The total force, the resultant of gravitational force and centrifugal force, yields a gravity vector that has magnitude and direction. The magnitude is called *gravity* (measured in gals), and the direction is called the*plumb line* or *vertical.* There are an infinite number of such equipotential surfaces, both above and below mean sea level, each with different forces of gravity that vary as a function of changes in the earth's mass and density.

Imagine for a moment that the earth is like an onion, with an infinite number of layers, each layer constituting a different equipotential surface on which the force of gravity is equal and the direction of the plumb line is always normal (perpendicular) to each equipotential surface. These layers undulate slightly with local variations in the mass and density of the earth, causing the plumb line to curve as shown at Figure A.1. Ultimately, water flows downhill because of higher gravity at lower elevations.

The *geoid* is that equipotential (level) surface of the earth's gravity field which, on average, coincides with mean sea level in the open, undisturbed ocean. In practical terms, the geoid is the imaginary surface where the oceans would seek mean sea level if allowed to continue into all land areas so as to encircle the earth. The geoid undulates up and down with local variations in the mass and density of the earth. The local direction of gravity is always perpendicular to the geoid.

A.5.2. **Ellipsoid**

Precise mapping and survey calculations are based on a mathematical surface called an *ellipsoid*—an ellipse of revolution, rotated around the (shorter) polar axis, which approximates the size and shape of the earth. As shown at Figure A.2, these ellipsoids have a semimajor axis *a* (distance from the ellipsoid's center to the surface along the equatorial plane) and a semiminor axis *b* (distance from the ellipsoid's center to the poles), where *b* is shorter than *a*. From *a* and *b*, the flattening *f* can be calculated [$f = (a - b)/a$].

For centuries, different ellipsoids were used around the world, chosen to best fit the terrain in different geographic areas. Until the 1970s, the United States used the Clarke 1866 ellipsoid that was not earth-centered (the ellipsoid's center did not coincide with the center of the earth). Other countries used different ellipsoids that better fit their local topography (e.g., International, Bessel, and Clarke 1880 ellipsoids used in Europe, Asia, and Africa), but none of those ellipsoids were earth-centered either.

Figure A.1 The elevation (orthometric height) of point P above the geoid equals the distance H measured along the plumb line passing through point P. Whereas the plumb line is perpendicular to each equipotential surface, these surfaces are not necessarily parallel to each other, causing the plumb line to curve. Each level has a different gravity potential (W), where W o is the potential of the geoid.

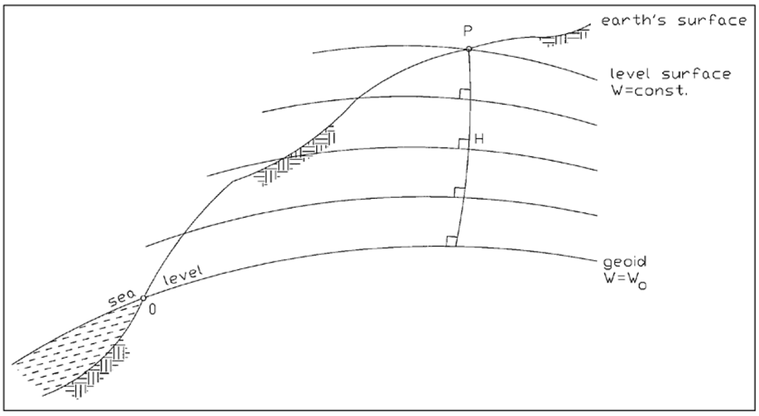
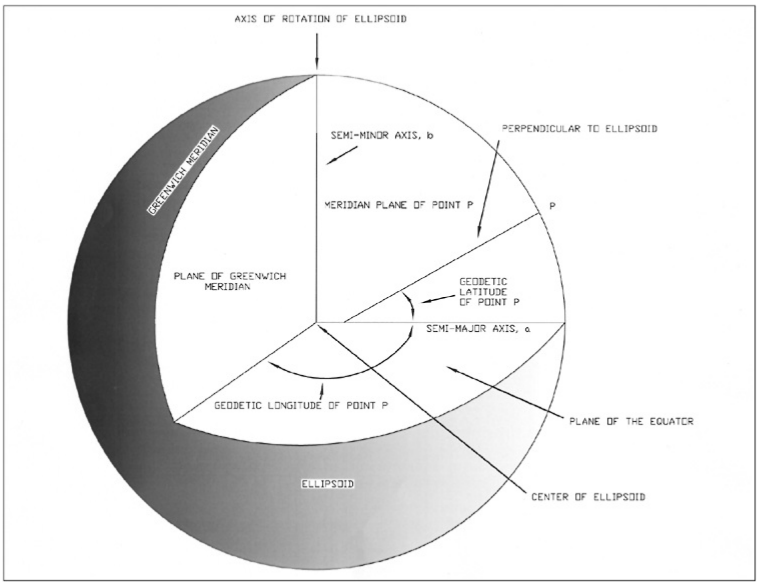


Figure A.2 This shows the semimajor axis a and semiminor axis b, which define an ellipsoid, as well as the geodetic latitude and geodetic longitude λ of point P. Note that the perpendicular to the ellipsoid does not pass through the center of the ellipsoid; if the earth were spherical instead of ellipsoidal, the perpendicular to the sphere would pass through its center.



With the advent of earth-orbiting satellites in the 1960s and 1970s, the United States was able to develop earthcentered ellipsoids, optimized to fit the entire world and not just individual continents or countries in isolation. One of the first was the World Geodetic System of 1972 (WGS 72), then the Geodetic Reference System of 1980 (GRS 80), and finally the World Geodetic System of 1984 (WGS 84). The defining parameters of the ellipsoids mentioned here are listed in Table A.3. These parameters are embedded in most standard GIS software used in the United States today.

Table A.3 Common Ellipsoids

ELLIPSOID	SEMIMAJOR AXIS (*a*)	SEMIMINOR AXIS (*b*)	FLATTENING INVERSE (1/*f*)
International	6,378,388 meters	6,356,011.9462 meters	285.05419
Bessel 1841	6,377,397.155 meters	6,356,078.9643 meters	299.15283
Clarke 1866	6,378,206.4 meters	6,356,583.8 meters	294.97870
Clarke 1880	6,378,249.145 meters	6,356,514.8696 meters	293.46500
WGS 72	6,378,135 meters	6,356,750.5 meters	298.25972
GRS 80	6,378,137 meters	6,356,752.3141 meters	298.25722
WGS 84	6,378,137 meters	6,356,752.3142 meters	298.25722

The numbers in Table A.3 are rounded for demonstration purposes and often contain many more significant figures than shown here. For example, the Geodetic Reference System 1980 (GRS 80) is the reference ellipsoid currently used for mapping in the United States. Its defining parameters are as follows:

■ Semimajor axis = equatorial radius =a = 6,378,137.00000 meters

■ Semiminor axis = polar radius =b = 6,356,752.31414 meters

■ Flattening =f (a –b)/a = 0.003352810681225

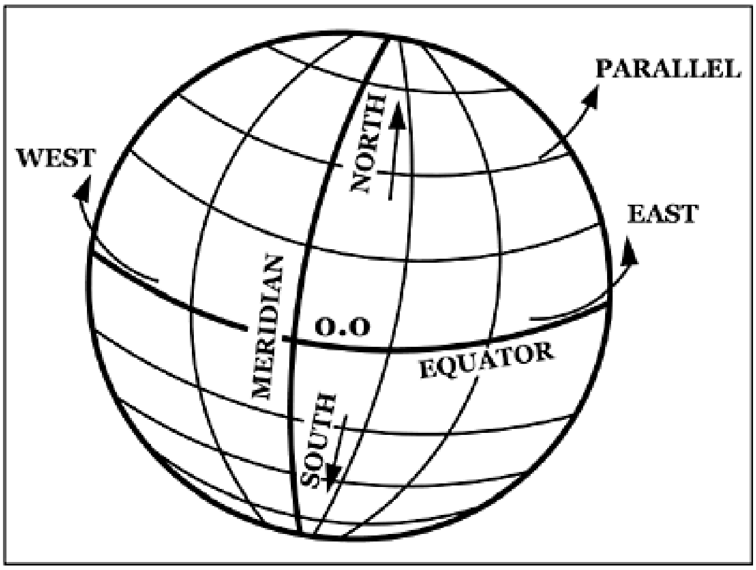
■ Inverse flattening = 1/f = 298.2572220972

The GRS 80 ellipsoid forms the basis for the current North American Datum of 1983 (NAD 83) and for all practical purposes is identical to the World Geodetic System of 1984 (WGS 84) ellipsoid. The Clarke 1866 ellipsoid formed the basis for the prior North American Datum of 1927 (NAD 27). For comparison purposes, the Clarke 1866 ellipsoid had a semimajor axis a of 6,378,206.4 meters, a semi-minor axis b of 6,356,583.8 meters, and inverse flattening 1/f of 294.9786982.

Figure A.3 shows the equator, parallels, and meridians used to understand geographic and geodetic coordinates.

Parallels are curved lines of equal latitude north or south of the equator, where the latitude is zero, and*meridians* are curved lines of equal longitude east or west of the prime meridian in Greenwich, England, where the longitude is zero. Figure A.3 clearly shows the *convergence of the meridians,* where the meridians are farthest apart at the equator and converge (get closer together) toward the north and south poles, where they merge.

Figure A.3 This shows parallels that define latitude north or south of the equator and meridians that define longitude east or west of the prime meridian passing through Greenwich, England. All meridians converge at the north and south poles. Latitude and longitude define the geographic coordinates of a point on the ellipsoid.



A.5.3. **Deflection of the Vertical**

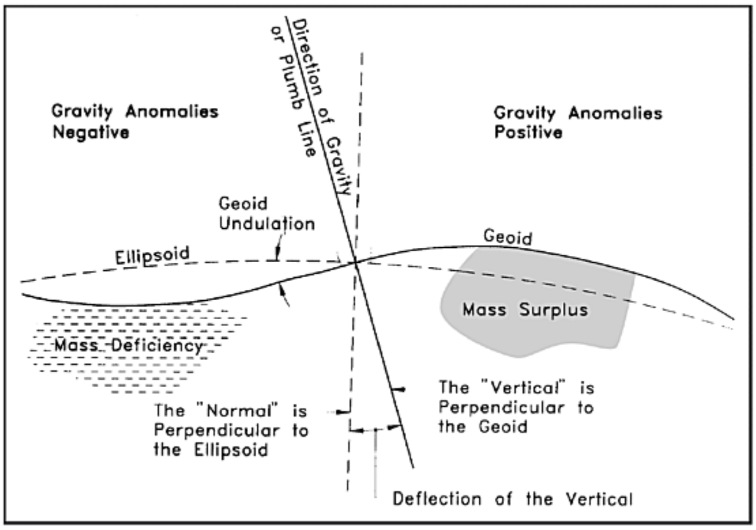
Conventional surveys typically start by leveling a survey instrument so that the instrument's horizontal line of sight is perpendicular to the local direction of gravity. Whether the surveyor realizes it or not, conventional surveys are subject to what could be called the "rules of gravity." However, unknown to almost everyone, the earth's mass anomalies cause variations in the local direction of gravity.

The equipotential surface known as the geoid is a smooth but undulating surface that always has the direction of gravity perpendicular to it. The *vertical,* a straight line that aligns with the plumb line (direction of gravity) at the earth's surface, is perpendicular to the geoid at that point and usually does not pass through the earth's center of mass. The *normal,* also a straight line, is perpendicular to the ellipsoid and usually does not pass through the center of the ellipsoid. The *deflection of the vertical* is the angle between the normal and the vertical. The *geoid undulation* is the distance, taken along a perpendicular to the ellipsoid, from the ellipsoid to the geoid. Because of undulations in equipotential surfaces, the *plumb line* curves and eventually joins all other plumb lines at the earth's center of mass. The earth's center of mass can now be mathematically defined as the point about which satellites orbit. College courses in physical geodesy, satellite geodesy, and geophysics provide greater details of such theories. Figure A.4 shows how changes in the earth's mass cause the geoid to undulate; it also shows that the ellipsoid's normal and the geoid's vertical are separated by the deflection of the vertical. The deflection of the vertical is never more than 2 arc-minutes anywhere on earth.

A.5.4. **Horizontal Datums**

A *geodetic datum* is a set of constants specifying the coordinate system used for geodetic control, that is, for calculating the coordinates of points on the earth. A horizontal datum is used to define horizontal coordinates, and a verti cal datum is used to define vertical coordinates. Horizontal datums require a reference ellipsoid, an origin, and an angular alignment.

Figure A.4 The geoid undulation above and below the ellipsoid and the deflection of the vertical illustrate why the rules of gravity, applicable to the undulating geoid, differ from the rules of geometry, applicable to the mathematically defined ellipsoid.



NAD 27, now obsolete, was a horizontal control datum for the United States that was defined by a location and an azimuth on the Clark 1866 ellipsoid, with origin at a survey station known as Meades Ranch (in Kansas) at which the geoid and ellipsoid were forced to coincide, as well as the normals (perpendicular to the geoid, and perpendicular to the ellipsoid). NAD 27 was used as the geodetic datum in the United States for most of the twentieth century. This artificially forced the deflection of the vertical (see Figure A.4) to be zero, and geodesists later realized that this warped the NAD 27 datum relative to earth-centered datums such as NAD 83 and WGS 84, both of which have origins at the center of mass of the earth. It was not until the 1980s that satellite orbits allowed geodesists to define the center of mass of the earth.

NAD 83 is the horizontal control datum now used for the United States, Canada, Mexico, and Central America, based on a geocentric origin and the GRS 80 ellipsoid. NAD is based on the adjustment of 250,000 points, including 600 satellite Doppler stations, which constrain the system to a geocentric origin.

NAD 83 was computed by the geodetic agencies of Canada (federal and provincial) and the National Geodetic Survey in the United States for several reasons. The horizontal control networks had expanded piecemeal since 1933 to cover much more of the countries, and it was very difficult to add new surveys to the network without altering large areas of the previous network. Field observations had added thousands of accurate electronic distance measuring equipment (EDME) baselines, hundreds of additional points with astronomic coordinates and azimuths, and hundreds of Doppler satellite–determined positions. It was also recognized that the Clarke Ellipsoid of 1866 no longer served the needs of a modern geodetic network.

NAD 27 was based on the Clarke 1866 ellipsoid and NAD 83 is based on the GRS 80 ellipsoid. The NAD 27 was computed with a single survey point, Meades Ranch, as the datum point, while the NAD 83 was computed as a geocentric reference system with no datum point. NAD 83 has been officially adopted as the legal horizontal datum for the United States. The computation of the NAD 83 removed significant local distortions from the network that had accumulated over the years, using the original observations, and made the NAD 83 much more compatible with modern survey techniques, including GPS surveys.

A.5.5. **Vertical Datums**

The National Geodetic Vertical Datum of 1929 (NGVD 29), now obsolete, was originally known as the Sea Level Datum of 1929. It was established for vertical control in the United States by the general adjustment of 1929. Mean sea level was held fixed at the sites of 26 tide gauges—21 in the United States and 5 in Canada. The datum was defined by the observed heights of mean sea level at the 26 tide gauges and by the set of elevations of all benchmarks resulting from the adjustment. A total of 106,724 kilometers of leveling was involved, constituting 246 closed circuits and 25 circuits at sea level.

The North American Vertical Datum of 1988 (NAVD 88) is the current official vertical datum in the United States, established by a minimum-constraint adjustment of the Canadian-Mexican-U.S. leveling observations. It held fixed the height of a single primary tidal benchmark, referenced to the new International Great Lakes Datum of 1985 (IGLD 85) local mean sea level height value, at Father Point/Rimouski, Quebec, Canada. Additional tidal benchmark elevations were not used due to the demonstrated variations in sea surface topography—that is, the fact that mean sea level is not the same equipotential surface at all tidal benchmarks.

NAVD 88 was computed for many of the same reasons as NAD 83. About 625,000 kilometers of leveling had been added to the NGVD since 1929. Thousands of benchmarks had been subsequently destroyed, and many others had been affected by crustal motion, postglacial rebound, and subsidence due to the withdrawal of underground fluids. Distortions amounting to as much as 9 meters had been seen as a result of forcing the new leveling to fit the NGVD 29 height values.

NGS develops and maintains the current NAVD 88. In addition, NGS provides the relationships between past and current geodetic vertical datums. However, another part of NGS's parent organization, the National Ocean Service (NOS), is the Center for Operational Oceanographic Products and Services (CO-OPS). CO-OPS publishes tidal benchmark information and the relationship between NAVD 88 and various water level/tidal datums (Mean Lower Low Water, Mean High Water, Mean Tide Level, etc.). The relationships to NGVD 29 are not published but may be calculated independently from specified tidal benchmark sheet links to the NGS database. Tidal benchmark information, water level/tidal datums, and their relationship to geodetic vertical datums are available at the CO-OPS website: www.co-ops.nos.noaa.gov.

A.5.6. **Map Projections**

Whereas precise survey calculations are based on a mathematical ellipsoid, survey calculations performed for land development purposes routinely assume the earth is flat, and they utilize a Cartesian coordinate system of 2-D (*x-y*) or 3-D (*x-y-z*) coordinates. This section explains how map projections are used to map the nearly spherical earth onto a two-dimensional map sheet, including cylinders and cones that are mathematically cut and laid flat. The next section will explain how State Plane Coordinate Systems are developed from these map projections tailored to best fit individual states within the United States.

To set the record straight from the very beginning, one cannot map a spherical surface onto a flat piece of paper without compromising one or more of the following: angles,

distances, or areas. Something has to give! In projecting from a spherical or ellipsoidal surface to a flat surface, the true shape of the features must change. However, map projections can be designed to preserve angles, distances, or areas, within limits, but not all three factors at once:

■ Conformal projections (e.g., transverse Mercator and Lambert conformal conic) maintain angular relationships and accurate shapes over small areas; such projections are used for navigation and general mapping.

■ Equal area projections (e.g., Peters and Albers equal-area conic) maintain accurate relative sizes; such projections are used for maps that show distributions or other phenomena where showing area accurately is important.

■ Equidistant projections (e.g., Equidistant conic or equirectangular) maintain accurate distances from the center of the projection or along given lines; such projections are used for radio and seismic mapping and for navigation.

■ Azimuthal projections (e.g., Gnomic and Lambert azimuthal equal-area) maintain accurate directions (and therefore angular relationships) from a given central point; such projections are used for aeronautical charts and other maps where directional relationships are important.

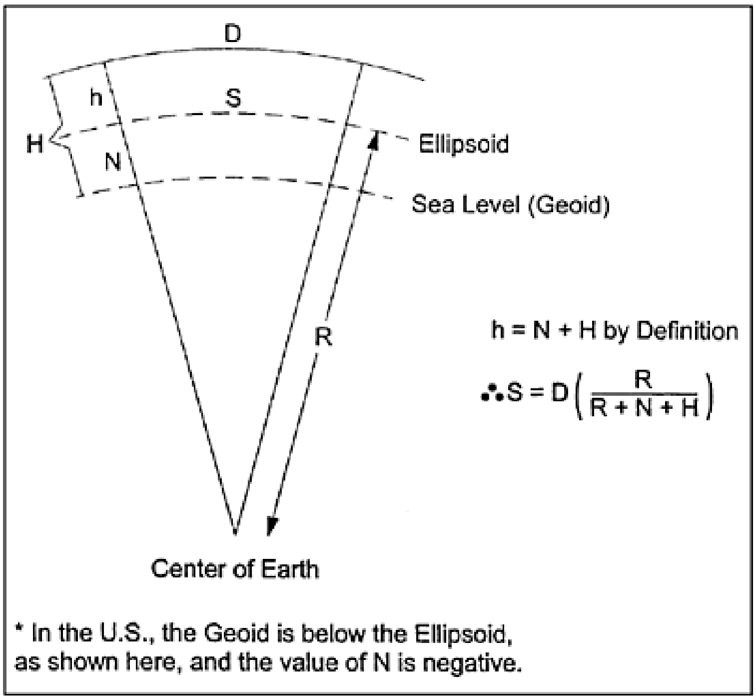
Because conformal projections are used with State Plane Coordinate Systems in the United States, subsequent discussions in this chapter focus primarily on the transverse Mercator and Lambert conformal conic projections, which are conformal projections.

All dimensions surveyed on the earth are changed proportionately when mapped at a reduced scale. Few points, however, are mapped exactly to the specified scale where the *scale factor* is exactly 1.0000000000, where the scale factor = (map distance)/(ground distance × scale). With Universal Transverse Mercator (UTM) coordinates, the scale factor is between 0.99960 and 1.00040. With State Plane coordinates, the scale factor is normally between 0.99990 and 1.00010. The scale factor on any map varies throughout the map and may vary in different directions at any given point.

The overall scale factor (also called the *grid scale factor*) is the product of the *elevation factor* and the *zone scale factor.* The elevation factor accounts for the elevation of the terrain being either above or below the ellipsoidal surface on which map projections are based. As shown in Figure A.5, the elevation factor equals $R/(R + H + N)$, where R = the mean radius of the earth (assume 6,372,000 meters), H = orthometric height or elevation above mean sea level (the geoid), and N = geoid height or separation from the ellipsoid (between −8 and −53 meters in the continental United States, i.e., always a negative number in the preceding equation).

As shown in Figure A.5, where the terrain is above the ellipsoid, the elevation factor will be slightly less than 1.

***Figure A.5** This explains the elevation factor that causes scaling errors as a result of elevations above or below the ellipsoid on which the map projection is based. Features scaled from a map (S) do not exactly match the surveyed ground distances (D) between those features, even after applying the published map scale because of variations in the elevations of mapped features.*

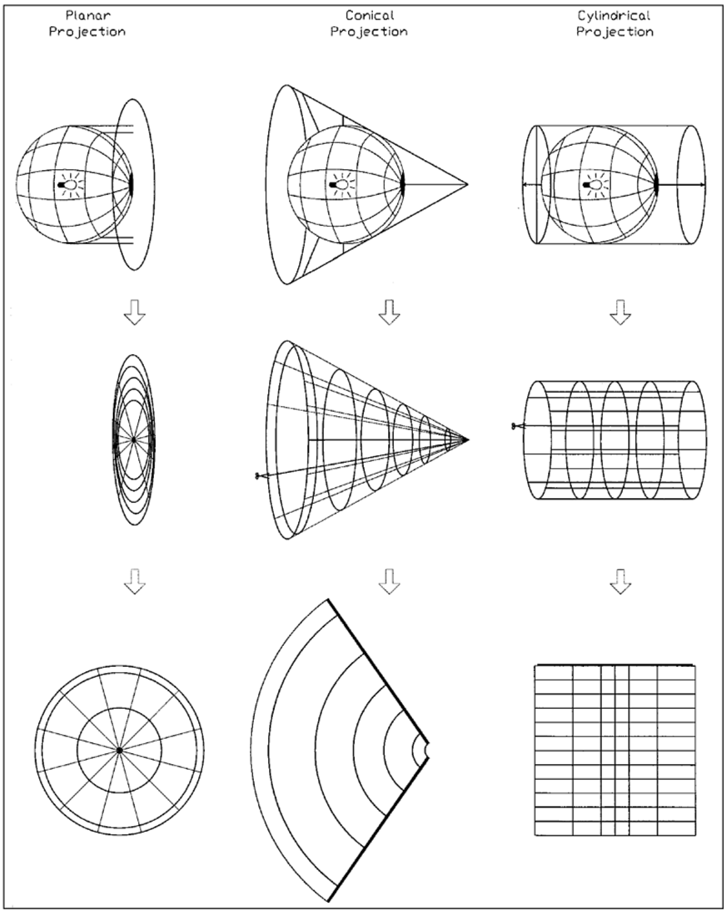


Alternatively, if the terrain was near sea level (below the ellipsoid), the elevation factor would be slightly greater than 1.

The zone scale factor accounts for the projection surface being either above or below the ellipsoid on which the map projection is based. The zone scale factor is exactly 1.000000000 only along a line where the projection surface is tangent to the ellipsoid. To better understand this concept, let's see how this works with the three standard projection surfaces: cylinders, cones, and planes.

As shown in Figure A.6 (*top*), imagine the earth to be a *theoretical glass ellipsoid* with all parallels, meridians, and natural and man-made features painted on its surface. Next, imagine an extremely large roll of light-sensitive paper encircling the earth, tangent along the earth's equator, starting and ending back at the Greenwich meridian after circling the earth, forming a huge cylinder with the axis of the cylinder passing through the north and south poles. Then, imagine a light at the center of the (glass) earth, projecting and mapping all painted features onto the paper, which, at full scale, unwrapped and laid flat, would be a huge map approximately 25,000 miles long. Earth features near the equator would be mapped to approximately 1:1 scale, but features north and south of the equator would be distorted, with scale factor errors increasing significantly with increased distance from the equator. The north and south poles would never project to the paper because they would remain inside the cylinder, no matter how long the cylinder is. At some point, users would choose to stop mapping this projection at high northern and southern latitudes because the cylinder would be too long, and distortions would be ridiculous.

Figure A.6 ***The earth's features, as well as meridians and parallels, can be projected onto a cylinder (top), onto a cone (center), or onto a plane surface (bottom). The cylinder and cone can then be laid flat to make a 2-D map of the 3-D ellipsoid. The lines or points of tangency can be changed to minimize map distortions for areas to be mapped. For example, with the cylindrical projection, instead of being tangent at the equator, the cylinder is commonly turned sideways to be tangent along a selected meridian. With the conical projection, instead of being tangent at the parallel shown, the shape of the cone is commonly changed so as to be tangent to any selected parallel. With the planar projection, instead of being tangent at the north pole, the plane can be moved to be tangent at any selected point on the earth. Wherever the projection surface is tangent to the ellipsoid, the scale factor of the map will be correct (1:1), with scale distortions worsening with increased distance from the lines or points of tangency.***



This map would be millions of times too large to fit on anyone's wall, but if this map was photographically or mathematically reduced in scale, it would approximate a map with the Mercator projection commonly used for world maps—maps that predictably show the area of Greenland to be enlarged/distorted compared to Greenland's actual size. The Mercator projection only maintains angular relationships and accurate shapes over small areas near the equator.

Next, imagine this cylinder of paper is turned sideways into a transverse position so that the axis of the cylinder is through the center of the earth and along the equator rather than through the poles, and imagine the cylinder is tangent to the earth along the 0° and 180° meridians. Now, only those features along a narrow north-south strip of land near these meridians would be mapped to scale, and other features would be mapped with scale factor errors that increase with distance east or west of these meridians. All east-west distances and areas would be mapped larger than they really are. This transverse cylinder can be rotated to be tangent along any number of different central meridians, each mapping features to scale along those meridians with scale factor errors increasing for features mapped to the east and west of each central meridian. The transverse Mercator projection only maintains angular relationships and accurate shapes over small areas near to the meridian along the line of tangency.

Next, imagine this transverse cylinder to have a smaller diameter than the earth, as shown inFigure A.7, so there are two *rings of intersection* (A-B and D-E) where the scale is truly 1:1. When the projected surface cuts through the earth, rather than being tangent to it, it is called a *secant projection*. Between these two rings of intersection (e.g., C-M), terrain features map smaller than they really are, and outside these two rings, terrain features map larger than they really are. By controlling the size of this transverse cylinder and limiting the east-west extents of areas mapped, users limit the distortions and scale factor errors to reasonable values. For example, by limiting the zones to 254 kilometers wide for a State Plane Coordinate System, scale factor errors will be between 0.9999 and 1.0001, or one part error in 10,000. Note in Figure A.7 (*top*) that these parallels and meridians are curved lines. When overlaid with an SPCS rectangular grid, *grid north* will differ from *true north* everywhere because of convergence of the meridians, except along the central meridian, which aligns with true north. The cylinder for the UTM projection cuts deeper into the ellipsoid, resulting in scale factor errors between 0.9996 and 1.0004.

National mapping agencies such as the U.S. Geological Survey utilize a UTM projection whereby there are 60 zones, each covering 6 degrees of longitude. Because these zones are much wider than 254 kilometers, a smaller cylinder is used to cut deeper into the earth with a secant projection so as to balance the areas that are larger and smaller than 1.0000. As a result, scale factor errors for the UTM projection are between 0.9996 and 1.0004, or one part error in 2500. The UTM projection is used for military maps that are universal, are used by U.S. forces and allies, and are not tailored to dimensions of individual countries. Whereas the UTM projection and UTM coordinates have appeal internationally and for large national mapping programs, their errors are considered to be too large for use in land development and construction projects in the United States where State Plane coordinates are used because of their greater accuracy and conformance to measurements surveyed on the ground.

Returning to Figure A.6 (*center*), imagine this same theoretical glass ellipsoid is draped with a huge cone of paper, tangent to the earth at the fortieth parallel (40 degrees north latitude). Using the light at the center of the earth to project features onto the cone, all features close to the fortieth parallel would be mapped close to true scale, but feature distortion would increase with greater distance north or south from the fortieth parallel. Next, imagine the cone's apex rising upward so that the cone is steeper and parallel to the ellipsoid at the thirtieth parallel. Now all features close to the thirtieth parallel would be mapped close to true scale, but feature distortion would increase with greater distance north or south from the thirtieth parallel. By raising or lowering the cone's apex, the cone can be made to be tangent to any desired parallel. For this reason, conic projections are preferred for mapping areas that are longer in the east-west direction where the land mass is closer to a single parallel, such as the state of Tennessee. The cone can also be unwrapped and laid flat so it forms a plane surface.

Next, imagine a secant cone cutting through the earth, as shown inFigure A.8, so that there are two *standard parallels* (A-B and C-D) where the scale is truly 1:1. This is a Lambert conformal conic projection used to establish the SPCS for many states in the United States. Between the standard parallels, terrain features are mapped smaller than they really are, and outside these standard parallels, terrain features are mapped larger than they really are. On a state-by-state basis, standard parallels can be moved north or south by changing the cone's apex and shape, making the cone steeper or shallower, and limiting the scale factor errors.

Returning again to Figure A.6 (*bottom*), it shows a planar projection where the theoretical glass ellipsoid is tangent at the north pole, the only point where the scale is truly 1:1; at all other locations, features would map larger than they really are. To limit such distortions, planar projections can also be secant, such as the polar stereographic projection widely used for mapping areas in the vicinity of the north and south poles. Planar projections do not need to be tangent or secant at the poles only; they could be tangent or secant anywhere on earth, but then for relatively small areas only. The Washington Dulles International Airport Coordinate System, described later, is one such use of a planar projection.

A.5.7. **State Plane Coordinate Systems**

Sometime around 1930, an engineer from the North Carolina state government approached the Coast and Geodetic Survey (C&GS, now NGS) and inquired about the possibil-

Figure A.7 Cylinder used for a transverse Mercator projection with two rings of intersection shown as A-B and D-E. Between these rings the scale is less than 1:1; outside these rings the scale is larger than 1:1. Meridians are mapped as slightly curved lines, except for the chosen central meridian, which is a straight line and where true north aligns with grid north. Parallels are mapped as slightly curved lines also, except at the equator.

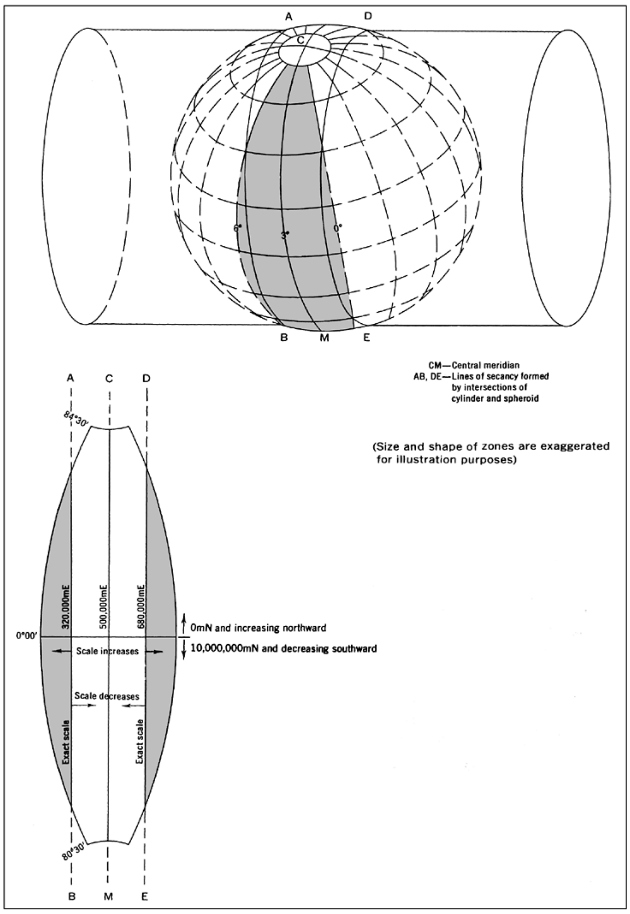
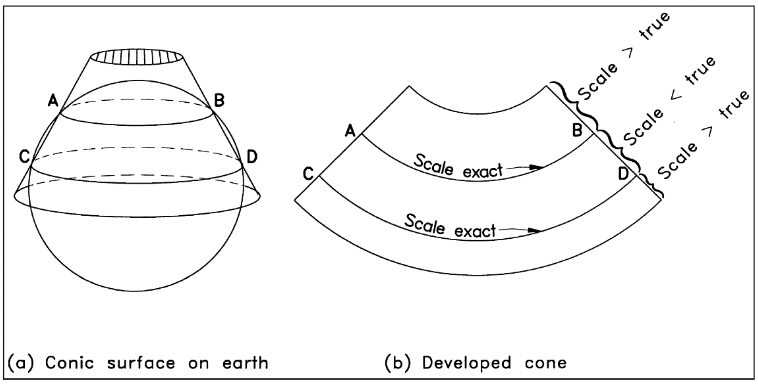


Figure A.8 Secant cone, cutting through the ellipsoid along two chosen standard parallels (A-B and C-D) used for a Lambert conformal conic projection. Between these two standard parallels the scale is less than 1:1; outside these standard parallels the scale is larger than 1:1. Meridians are mapped as non-parallel straight lines that will divert from grid north except along the central meridian chosen for the map projection; grid north aligns with true north only along the chosen central meridian. Parallels are mapped as slightly curved lines (segments of circular lines) everywhere.



ity of using simple techniques to survey and map the entire state with rectangular grid coordinates. The engineer wanted to ignore the curvature of the earth and assume the earth's surface to be a flat plane; such simple techniques are called *plane surveying techniques*. The C&GS explained that it's impossible to flatten the curved surface of the earth into a plane without distorting the surface in one way or another, but this inquiry led to a cooperative venture between the C&GS and the North Carolina state government to build a North Carolina spatial coordinate system with *minimal* distortion so that the scale factor was as close as possible to 1:1. In 1933 this cooperative venture produced the North Carolina Coordinate System. In less than 12 months, the North Carolina system had been copied for all of the remaining states, and the SPCS was born. Today the SPCS covers all 50 states.

State Plane Coordinate Systems were developed to keep scale factor errors smaller than one part in 10,000. With variations tailored for each state, Transverse Mercator projections are used as the basis for State Plane coordinates for *tall states* that are longer in the north-south direction, and Lambert conformal conic projections are used as the basis for State Plane coordinates for *wide states* that are longer in the east-west direction. To limit distortions to within this range, the C&GS (now NGS) limited the north-south dimension of each Lambert conformal conic zone and the east-west dimension of each transverse Mercator zone to 254 kilometers. If tall states are wider than 254 kilometers, or if wide states are taller than 254 kilometers, they would normally have two or more SPCS zones. Regardless of projection, each SPCS zone requires a central meridian with origin (ϕ_0, λ_0) and *false easting* (E_0) and *false northing* (N_0) coordinates for the origin so that all easting and northing coordinates are positive numbers within a normal range of values. Regardless of whether the transverse Mercator or Lambert conformal conic projection is used, grid north is different from true north at all locations except for points along the central meridian of each SPCS zone. Only along the central meridian of these two projections does true north align with grid north.

State Plane Coordinate Systems are not projections, but they rely on projections to minimize distortions on SPCS grids. Let's look at two examples.

Indiana, for example, is a *tall state* that is longer in the north-south direction, and its east-west extent is between 254 and 508 kilometers, requiring two transverse Mercator zones (east zone 1301 and west zone 1302):

■ Indiana's east zone has its central meridian at 85.66666666666667 degrees west longitude where its scale factor is 0.9999666666666667. The latitude of the origin is 37.5 degrees north latitude, and its longitude is the same as that of the central meridian for the east zone. The false easting for the east zone's origin is 328083.3333333333 U.S. survey feet, and the false northing for the origin is 820208.3333333333 U.S. survey feet.

■ Indiana's west zone has its central meridian at 87.08333333333333 degrees west longitude where its scale factor is 0.9999666666666667. The latitude of the origin is 37.5 degrees north latitude, and its longitude is the same as that of the central meridian for the west zone. The false easting for the west zone's origin is 2952750.0000000000 U.S. survey feet, and the false northing for the origin is 820208.3333333333 U.S. survey feet.

Similarly, Virginia is a *wide state* that is longer in the east-west direction, and its north-south extent is between 254 and 508 kilometers, requiring two Lambert conformal conic zones (north zone 4501 and south zone 4502).

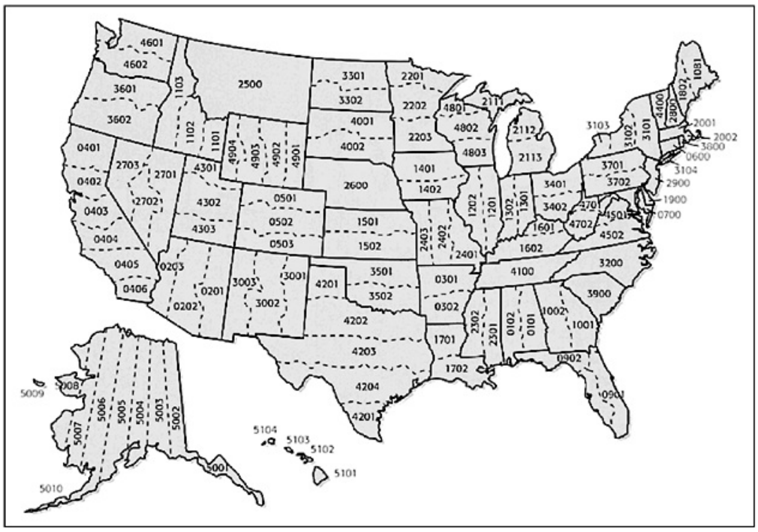
■ Virginia's north zone has its central meridian at 78.5 degrees west longitude. The two standard parallels (where the scale factor is exactly 1) are: 38.03333333333333 degrees and 39.2 degrees north latitude. The latitude of the origin is 37.6666666666667 degrees north latitude and its longitude is the same as that of the central meridian for the north zone. The false easting for the north zone's origin is 11482916.66666666 U.S. survey feet, and the false northing for the origin is 6561666.666666666 U.S. survey feet.

■ Virginia's south zone has its central meridian at 78.5 degrees west longitude. The two standard parallels (where the scale factor is exactly 1) are at 36.76666666666667 degrees and 37.96666666666667 degrees north latitude. The latitude of the origin is 36.33333333333334 degrees north latitude and its longitude is the same as that of the central meridian for the south zone. The false easting for the south zone's origin is 11482916.66666666 U.S. survey feet, and the false northing for the origin is 3280833.333333333 U.S. survey feet.

Generally, the boundaries between state plane zones within a state follow county boundary lines so that entire counties are within a single SPCS zone. This makes the north-south boundary between east and west zones and the east-west boundary between north and south zones an irregular line, as shown in Figure A.9. The only exception to this county boundary rule is in Alaska, where counties do not exist. Alaska also has one SPCS zone that uses neither the Lambert conformal conic nor the transverse Mercator projections, instead using an oblique Mercator projection for a portion of the coast that runs from the southeast to the northwest. This is the only exception to the two standard map projections used nationwide with State Plane Coordinate Systems.

Florida uses both transverse Mercator and Lambert conformal conic projections for its SPCS. Florida's East zone is a transverse Mercator zone that covers eastern Florida, including its Atlantic coast. Florida's West zone is a transverse Mercator zone that covers southwestern Florida, including its western coast on the Gulf Coast up through Levy County. Florida's North zone is a Lambert conformal conic zone that covers northern Florida, including the southern coast on the Gulf along the Florida panhandle.

Figure A.9 ***With few exceptions (e.g., Montana and Nebraska), most states have adopted a State Plane Coordinate System to keep scale factors between 0.99990 and 1.00010 so that scaling errors will be 1:10,000 or less. Generally speaking, states longer in the north-south direction utilize one or more transverse Mercator zones, whereas states longer in the east-west direction utilize one or more Lambert conformal conic zones. Zone boundaries normally curve to coincide with county boundaries in order to keep entire counties within a single zone.***



There are a total of approximately 120 SPCS zones throughout the United States, and the total varies, as some states decide to adopt a single SPCS zone rather than multiple zones. Montana and Nebraska, for example, chose to retain a single SPCS zone even though they are taller than 254 kilometers; such states accept scale factor errors greater than 1 in 10,000.

It is important for planners to understand two additional facts about State Plane coordinates:

■ Some states are metric, with State Plane coordinates in meters, whereas other states use English measurements in feet.

■ Most counties use the U.S. survey foot as the unit of measurement while other counties in the same state use the International standard foot. The difference is approximately 1 part in 500,000 and is negligible in some cases. Still it is important for everyone to agree on the unit of measure for each project.

When converting between metric and English units in an SPCS, it is important to use all decimal places in the conversion factor. The author is aware of one project that cost an engineering firm over $1 million to correct errors in converting from meters to feet because the conversion factor 3.28 was used instead of 3.2808333, causing errors in excess of 10 feet in coordinates because of this oversight.

In some cases, users cannot accept errors amounting to 1 part in 10,000 as per the SPCS. One such case involved the Washington Dulles International Airport Coordinate System, designed by the author's company (Dewberry), in which a central meridian was supposed to align with the centerline of the main north-south runway, and for which the scale factor errors had to be less than 1 part in 10,000,000, that is, scale factor errors between 0.9999999 and 1.0000001 anywhere on the airport property. During the conduct of this contract, and after extensive surveys, the author demonstrated to the client that the runway was not constructed true north and south, as initially believed (and as preferred by pilots), but was aligned approximately to grid north on the Virginia SPCS. Whereas a survey monument at the north end of the runway was only 4 inches off from grid north (relative to a monument at the southern end of the runway on the Virginia SPCS grid), it is more than 130 feet off from true north; that is, it is misaligned from true north by approximately 39 arc-seconds (nearly 2/3 of 1 degree). In developing the Dulles Airport Coordinate System, the author had to abandon plans to use a grid based on the transverse Mercator projection for which the central meridian must be true north—instead resorting to a secant, planar projection with origin at the south end of the main runway, and making the Airport Coordinate System much more complicated than initially planned. The client was surprised to learn of this significant difference. The author has never learned whether this runway alignment was intentional or unintentional when initially surveyed and constructed, but the fact remains that *it is entirely possible for an airport runway to be misaligned by the difference in azimuth between grid north and true north if the design engineers and surveyors do not recognize the difference between these terms.* With visual flight rules, pilots can compensate for such misalignments, but the day may come when automated landing procedures could bring a plane in for landing on the wrong bearing.

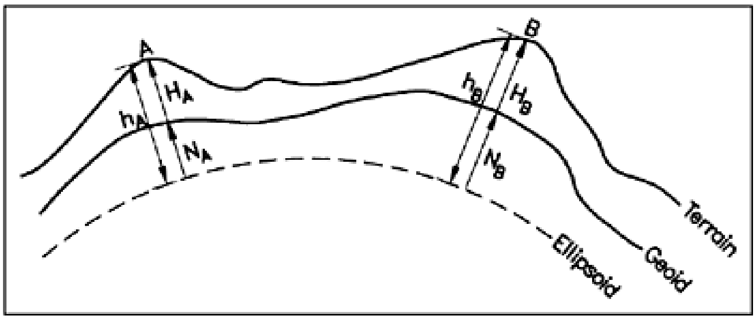
A.5.8. **Elevations and Heights**

Figure A.10 shows the physical terrain, with elevations above the *ellipsoid* and elevations above the *geoid.* These elevations are technically called *heights*. Elevations above the ellipsoid, called *ellipsoid heights* (h), follow the rules of geometry (as do GPS surveys), whereas elevations above the geoid, called *orthometric heights* (H), follow the rules of gravity (as do conventional surveys with differential levels, theodolites, and total stations). Elevation data can be presented as both ellipsoid heights and orthometric heights, but the popular term *elevation* normally refers to the orthometric height of a point, that is, its height above the geoid as measured along the plumb line between the geoid and a point on the earth's surface, taken positive upward from the geoid.

The *orthometric heights* (H_A and H_B) of points A or B above the geoid are measured along the *plumb lines* (direction of gravity) at those points; the *ellipsoid heights* (h_A and h_B) of points A or B are measured along the ellipsoid *normal* (perpendicular to the ellipsoid); the *geoid heights* or *undulations* (N_A and N_B) are the differences between the ellipsoid heights and the orthometric heights at A and B; and the *deflections of the vertical* (see Figure A.4) are the angular differences between the ellipsoid normal and the plumb line at A and B.

Ellipsoid heights from GPS surveys are converted to traditional orthometric heights by applying the geoid height, using the latest geoid model available from the NGS. This is currently Geoid03. The standard geodetic conversion formula is: $H = h - N$.

Figure A.10 This shows the three reference surfaces used in mapping (physical terrain, geoid, and ellipsoid) and the primary parameters used for mapping elevations: orthometric height H above the geoid, ellipsoid height h above the ellipsoid, and geoid height N or geoid undulation between the ellipsoid and geoid. For application in the internationally used geodetic equation H = h − N, N is always a negative number in the continental United States. Whereas this figure shows the geoid above the ellipsoid as in much of the world, in the United States, the geoid is below the ellipsoid, causing N to be a negative number.



As indicated, the *geoid* is an equipotential surface (equal force of gravity throughout) equivalent to that at mean sea level. There are other equipotential surfaces above and below the geoid that have lesser and greater forces of gravity than along the geoid. The higher above the earth's surface one theoretically goes, the lesser the force of gravity. The closer to the center of the earth one theoretically goes, the more confusing it becomes because there are masses on all sides pulling in every conceivable direction. The earth's center of mass is the balancing point about which satellites orbit.

Elevations (orthometric heights) are derived mathematically but follow gravimetric principles. Geodesists sometimes describe the geoid as a smooth but undulating surface that defines where mean sea level would go if the land mass theoretically got out of the way, perhaps forming narrow trenches for the water to pass inland while retaining the mass excesses and deficiencies that cause the geoid to undulate. Even the ocean surface undulates because of nonhomogeneous variations in the mass of the earth beneath the ocean waters.

The first and second editions of *Digital Elevation Model Technologies and Applications: The DEM Users Manual,* edited by the author and published by the ASPRS, includes an entire chapter on vertical datums, including the National Geodetic Vertical Datum of 1929 (NGVD 29), the North American Vertical Datum of 1988 (NAVD 88), the International Great Lakes Datum of 1985 (IGLD 85), and numerous tidal datums including Mean Higher High Water (MHHW), Mean High Water (MHW), Mean Sea Level (MSL), Mean Low Water (MLW), Mean Lower Low Water (MLLW), and other tidal values including Mean Tide Level (MTL), Diurnal Tide Level (DTL), Mean Range (Mn), Diurnal High Water Inequality (DHQ), Diurnal Low Water Inequality (DLQ), and Great Diurnal Range (Gt). Rather than explaining in detail the differences, the following major points are summarized:

- There are a large number of vertical datums used in the United States.
- There can be large differences between these various datums—as much as tens of meters in elevations.
- Users must know which vertical datum their products are referenced to.
- Users should not combine datasets that are referenced to different vertical datums.
- Published local mean sea level heights (LMSL) and published national vertical heights (NAVD 88) are not the same.
- New technology heights may not be compatible with old published heights.
- Areas subject to subsidence are particularly suspect.
- Users should reference their products to the latest National Vertical Datum, NAVD 88, because of traceability and the likelihood of being able to transfer from one datum to another.

A.5.9. **Major Points to Remember about Geodesy**

■ The reference ellipsoid most commonly used in the United States is the Geodetic Reference System of 1980 (GRS 80).

■ The horizontal datum most commonly used in the United States is the North American Datum of 1983 (NAD 83); NAD 83 coordinates are not the same as NAD 27 coordinates. Similarly, the official vertical datum in the United States is the North American Vertical Datum of 1988 (NAVD 88). Users must know what datums their products are referenced to and not combine datasets that are referenced to different datums.

■ Map projections always have scale factor errors between distances measured and scaled on maps and distances measured on the ground. Most State Plane Coordinate Systems have scale factor errors limited to 1 part in 10,000, but Montana and Nebraska are exceptions, with larger scale factor errors.

■ State Plane Coordinate Systems utilize rectangular grids of eastings and northings; grid north is almost always different from true north and could cause the misalignment of features such as airport runways if not taken into account.

■ Some State Plane Coordinate Systems are based on metric units, whereas others are based on English units. Furthermore, some counties use the U.S. survey foot, while other counties in the same state use the International standard foot with slightly different dimensions.

■ Measurement units and full conversion factors must be clearly defined.

■ Ellipsoid heights from GPS surveys are not the same as orthometric heights from traditional land surveys.

■ Areas subject to subsidence require special considerations for which the National Geodetic Survey (NGS) state advisor should be consulted for advice.

A.6. **PHOTOGRAMMETRY**

A.6.1. **Photogrammetric Applications**

Photogrammetry is an art, a science, and a technology. Aerial photogrammetry uses stereo aerial photographs or stereo digital images to create planimetric and/or topographic maps of features visible on the imagery and to determine the relative location of points, lines, and areas for determination of distances, angles, areas, volumes, elevations, sizes, and shapes of mapped features. Photo interpretation of such imagery deals with recognizing and identifying objects on the imagery and judging their significance through careful and systematic analyses.

Stereo photographs or stereo images are those taken of the same area on the ground but viewed from two perspectives. Aerial stereo photographs are commonly flown with each image having a 60 percent overlap with the preceding and subsequent images; this enables 60 percent of each photograph to overlap the same area shown on the preceding photograph, 60 percent of each photograph to overlap the same area shown on the subsequent photograph, and 10 percent of each photograph to appear on three successive photographs. With stereoscopic viewing, much greater depth perception can be obtained. Stereoscopic viewing enables the formation of a three-dimensional stereomodel for viewing a pair of overlapping photographs, making accurate 3-D measurements and mapping elevations in addition to planimetric detail.

Terrestrial photogrammetry follows the same general principles as aerial photogrammetry except that special metric cameras are mounted on tripods and the line of sight is generally horizontal rather than vertical, but still photographing the same area from two different perspectives.

A large percentage of photogrammetric applications have traditionally pertained to topographic mapping, at various scales, for planning and design of transportation features (highways, railroads, rapid transit systems, airfields, bridges, culverts), pipelines, aqueducts, transmission lines, flood control structures, river and harbor improvements, urban renewal projects, shopping malls, and housing areas, for example. Two newer photogrammetric products, *digital orthophotos* and *digital elevation models,* are now often used in combination to replace traditional topographic maps. An orthophoto is an aerial photograph that has distortions removed, has a uniform scale throughout, and has the metric properties of a planimetric map; however, unlike planimetric maps that show features by using lines and symbols, orthophotos show the actual images of features, making them easier to interpret. A digital elevation model (DEM) is an array of points with *x-y-z* coordinates. DEMs model the 3-D topography or shape of the terrain from which contours, cross sections, and profiles can be computed. Orthophotos and DEMs are widely used in all fields where maps are used, but because they are in digital form, they are ideal for use in geographic information systems (GISs).

Photogrammetry is often used to supplement land surveys. Although aerial photographs don't show boundary lines, aerial photos and/or orthophotos can be used as rough base maps for relocating existing property boundaries. If the point of beginning or any corners can be located with respect to ground features that can be identified on aerial photos, an entire parcel can be plotted on the orthophoto from the property description. All corners can then be located on the photo in relation to identifiable ground features, which, when located in the field, greatly assist in finding the actual property corners. Aerial photos can also be used in planning ground surveys. Through stereoscopic viewing, areas can be studied in 3-D; access routes to remote areas can be identified, and surveying lines of least resistance through difficult terrain or forests can be found. Photogrammetrists map areas without actually setting foot on the ground, avoiding the need to gain access to private land for surveys, avoiding surveys in wetlands and terrain where ground mobility is difficult, and avoiding surveys along highways where land surveys are either unsafe to surveyors or slow the flow of traffic while surveys are in progress.

Photogrammetry is especially important for highway planning and design. Aerial photographs are used in preliminary planning for selection of corridors for new routes. Large-scale topographic maps, contours, and/or DEMs are used in final design. Cross sections are used to compute cut and fill for earthwork quantities in contracts. Plan profile sheets of highway plans are prepared from aerial photos and/or orthophotos. Partial payments and final payments are often calculated from photogrammetric measurements made on photographs acquired during various stages of construction. Information collected from photogrammetry is normally compiled in computer-aided design and drafting (CADD) formats such as AutoCAD .dwg or MicroStation .dgn formats commonly used for highway design, reducing costs and enabling better overall highway design and construction. The same is true for other land development projects such as construction of new housing areas and commercial and industrial areas. Other photogrammetric applications include the mapping of building footprints and the preparation of tax maps, soil maps, forest maps, geological maps, planning and zoning maps, and land use/land cover maps. Photogrammetry is used in the fields of astronomy, architecture, archaeology, geomorphology, oceanography, hydrology and water resources, conservation, ecology, and mineralogy, enabling the harsh outdoor environment to be surveyed, mapped, and analyzed in the comforts of an office.

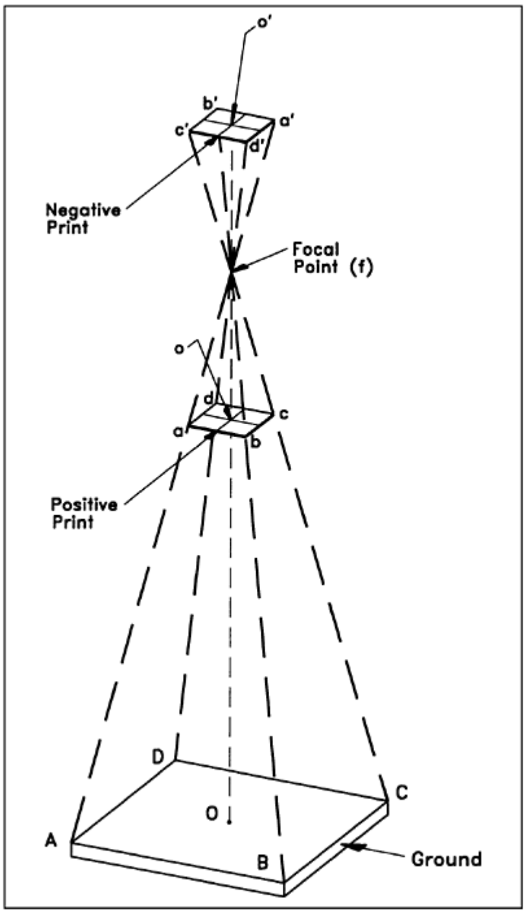
A.6.2. **Photogrammetric Equipment**

Although rapidly being replaced by digital cameras for large mapping projects, the traditional aerial film camera is still the most economical for mapping of typical land development projects. Except for oblique imagery (discussed later), used for special applications, most aerial photos are vertical, meaning the cameras are intended to point straight down. Photographs are rarely truly vertical because of unavoidable roll and pitch of the aircraft in flight, but near-vertical photographs are still the goal.

Cameras with 6-inch focal length are the most common for general-purpose applications, balancing the preference for long focal lengths to maximize horizontal accuracy and short focal lengths to maximize vertical accuracy of topographic maps. Figure A.11 shows how rays of light from points A, B, C, and D on the ground are imaged through the lens focal point to be recorded as points a', b', c', and d' on the film *negative. Positives* of those same *negatives* display those points at a, b, c, and d on a film positive or contact print.

Whether acquired from traditional aerial film cameras or modern digital cameras, aerial imagery records perspective views of the terrain. Features that are taller or closer to the camera appear larger than features that are farther away. Thus, aerial imagery inherently has *relief displacement* that causes the images of elevated features to be displaced outward from the center of each photo or image. Furthermore, aerial photographs are rarely truly vertical (i.e., looking straight down at the ground); there is normally some degree of roll, pitch, and yaw of the aircraft that causes photographs to be slightly oblique. Thus, aerial photographs also inherently have *tilt displacement* that causes some features to be tilted outward while other features are tilted inward relative to the center of an aerial photograph.

Figure A.11 Vertical aerial photo geometry.



The objective of traditional photogrammetric mapping is to remove all relief displacement and tilt displacement in order to map the terrain with an orthographic projection as though every feature on the ground were viewed looking straight down from infinity. Figure A.12 shows how aerial photography is planned to achieve the desired scale of photography and the desired forward overlap of 60 percent or more between consecutive photographs. With *stereo* coverage (same terrain photographed from two different perspectives), stereo photogrammetric techniques can be used to map features in three dimensions. Figure A.13 shows how 3-D coordinates of points on the ground are mapped from 2-D coordinates of those points as imaged on stereo photographs. Automated image correlation is commonly used to map either the digital surface model (DSM) or the digital terrain model (DTM). The DTM requires additional editing to remove elevations of vegetation and man-made features above the bare-earth terrain. Then the DTM is used as the terrain model over which a single aerial photograph can be draped in order to produce a digital orthophoto of the terrain. The generic digital elevation model (DEM) is generally synonymous with the DTM, although a DEM usually has regularly spaced elevation points, whereas a DTM has irregularly spaced elevation mass points and breaklines.

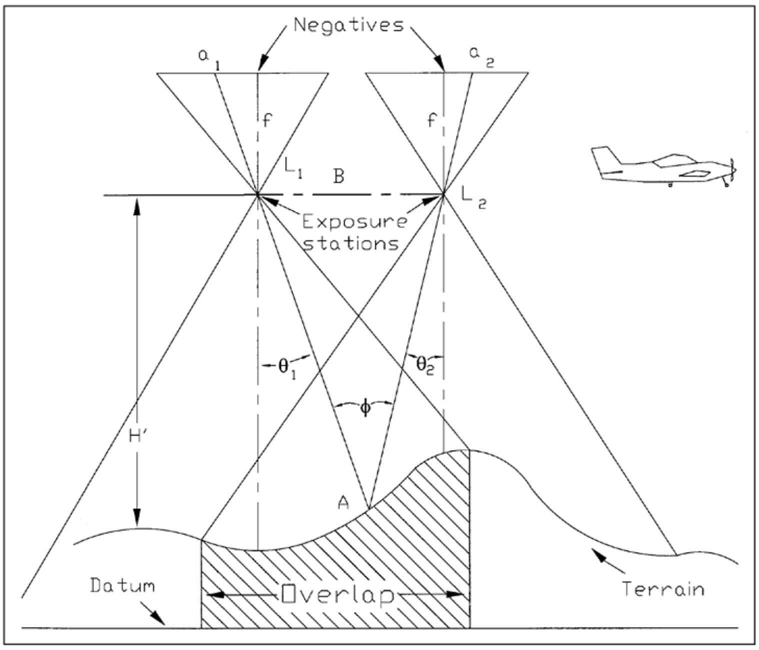
There are three distinct generations of photogrammetric equipment that might be used for planimetric and/or topographic mapping for land development projects.

Analog Plotters. Used with hard-copy aerial photos only, analog plotters are optical/mechanical photogrammetric instruments that physically replicate (at reduced scale) the spatial geometry that existed when the aerial photographs were taken. Analog plotters use different techniques to enable the photogrammetric map compiler to see the stereo images in 3-D, determine the elevation of points on the ground, and plot contour lines. Although generally considered to be obsolete in the United States, analog plotters are still used by some photogrammetric companies for small mapping projects such as those for land development projects. Because of limited capabilities to correct for nonlinear distortions, analog plotters are the least accurate and the least efficient.

Analytical Plotters. Also used with hard-copy aerial photos, analytical plotters are computerized photogrammetric instruments that mathematically replicate the spatial geometry that existed when the aerial photographs were taken. Special optics enable the photogrammetric map compiler to see simultaneously the stereo images in 3-D and place a floating dot on the ground to determine the elevation of any point seen in stereo. When set for a specific elevation, the floating dot can be moved along the ground to trace a contour line at uniform elevation. Analytical plotters are very accurate and excellent for land development mapping projects.

Soft-Copy Workstations. Used with digital imagery only (including scanned film), soft-copy photogrammetric workstations mathematically replicate the spatial geometry that existed when the aerial imagery was acquired. Polarized images or other sophisticated techniques enable the photogrammetric map compiler to see simultaneously the stereo images in 3-D on a computer screen and determine the elevation of any point seen in stereo. Looking at the computer screen with polarized glasses (or alternative procedures), the compiler keeps a floating dot on the ground and traces contour lines. Many other photogrammetric procedures are automated, to include the automated production of DSMs as well as pass points used in aerial triangulation. Hard-copy photos are scanned at resolutions between 7 and 25 micrometers, typically, to convert them into high-resolution digital images. Soft-copy workstations yield high-accuracy planimetric and topographic maps, as well as other photogrammetric products such as digital orthophotos, DSMs, and DTMs.

Figure A.12 The aircraft flies at a preplanned elevation (H') above mean terrain in order to obtain the desired scale of photography. The base (B) between exposure stations (L 1 and L 2) is planned to achieve ≥60 percent overlap between exposures, allowing the terrain in the hashed area to be mapped in stereo. Stereo photogrammetry converts images from a perspective projection into an orthographic projection, as though looking straight down from infinity. The larger the angle above, the more accurate are the mapped elevations.



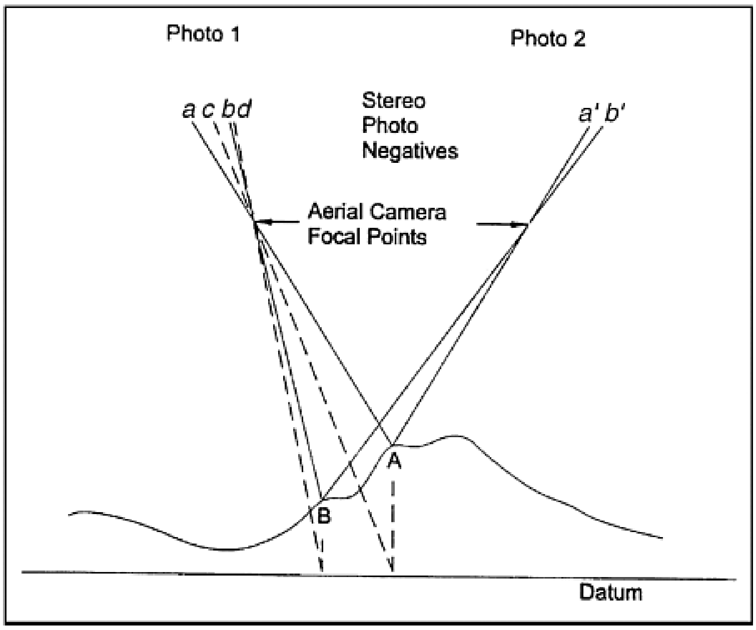
A.6.3. **Aerial Triangulation**

Aerial triangulation (AT) has traditionally been required to control a block of aerial photographs, to force mapped coordinates to fit surveyed ground control, and also to control other mapped coordinates for which surveyed ground coordinates are unknown. This traditionally has required a large number of photoidentifiable, surveyed ground control points. Photogrammetrists often use painted symbols or paneled survey targets in the shape of an X, T, or Y, prepositioned and surveyed prior to acquisition of aerial photography, to ensure that there are sufficient photoidentifiable points with known coordinates. When there are already ample photoidentifiable features, photogrammetrists may instead rely on photo points that naturally appear at desirable locations on the overlapping photography and survey those points after the photography has been flown and the film developed.

One of the objectives of AT is to determine six exterior orientation parameters for each of the aerial photographs to be used; these six parameters include *x, y,* and *z* coordinates (in airspace) of the focal point of the lens when each photograph was taken, as well as the roll, pitch, and yaw of the aircraft around the *x* axis (direction of flight), *y* axis (cross direction), and *z* axis (vertical axis).

When these photo control points are surveyed from multiple control monuments surrounding the area, the AT solution may be weakened by the fact that multiple control monuments are often inconsistent relative to each other; then AT residual errors are high. It is preferred that GPS procedures be used, with a single pair of GPS base stations of highest accuracy in order to obtain consistent results, a strong AT solution, and maps that will be more accurate as a result. When this is done, then the AT residual errors are small. Experienced photogrammetrists know how to examine these residuals to determine whether the survey control is strong or weak. Fully analytical aerial triangulation (FAAT) is generally considered to yield the highest accuracy and lowest residuals.

Figure A.13 The horizontal and vertical coordinates of A and B are mapped by the intersection of light rays through points a and b (left photo) and a' and b' (right photo) as projected through their respective focal points. As shown here, points a and b in the left photo should be at c and d in order to be orthographically correct when mapped relative to the mapping datum. Such photogrammetric corrections are not applied, pixel by pixel, to digital orthophotos; instead, a single image is draped over a digital elevation model of the bare-earth terrain.



Figures A.14 and A.15 both show an obvious variation in the roll, pitch, and yaw of the aircraft between the left and right photos. Whether variations in elevations or orientations are major or minor, a strong AT solution is required to determine the six exterior orientation parameters for each photograph. If the AT solution is strong, rays of light from the left photo will cleanly intersect with rays of light from the right photo, as shown in Figure A.14. However, if the AT solution is weak, rays of light from the left photo will not intersect with rays of light from the right photo, as shown in Figure A.15. With a weak AT solution, the 3-D coordinates of mapped points have larger error ellipses and uncertainty in derived coordinates. Therefore, a strong AT solution is the key to accurate topographic mapping.

Since the 1990s, aerial survey firms have regularly used airborne GPS and inertial measurement unit (IMU) technology to simplify the AT process, and for mapping projects over inaccessible or environmentally sensitive terrain. Airborne GPS is capable of measuring the *x, y,* and *z* coordinates directly in airspace, and IMUs are capable of measuring the roll, pitch, and yaw directly as each photograph is exposed. Some ground control points are still used for the AT process, but the numbers are reduced significantly when airborne GPS and IMU technology is properly used. However, this advanced technology is more relevant to large mapping projects than to small projects for land development applications.

A.6.4. **Photo Scale**

Photo scale is a critical factor in planning aerial photography. Photo scale is a function of the flying height above the terrain and the focal length of the mapping camera. With a mapping camera having a given focal length, the required photo scale normally establishes the flying height to be used. Subsequent scale variations in a photograph or between successive photographs are caused by variations in the terrain elevation, by variations in flying heights, or both.

Figure A.16 depicts two photographs taken over terrain having an average elevation of 400 feet above the datum and a range in elevation from 175 to 600 feet. In each case, the average photo scale is 1 inch = 200 feet.

With a 6-inch focal length camera, the flying height would be 1600 feet above mean terrain (amt). At an elevation of 175 feet, the scale would be 6-inch (0.5-feet) divided by (1600 – 175) feet, or 1 inch = 238 feet. However, at an elevation of 600 feet, the scale would be 0.5-feet divided by (1600 – 600) feet, or 1 inch = 167 feet. This is a relatively large variation in scale.

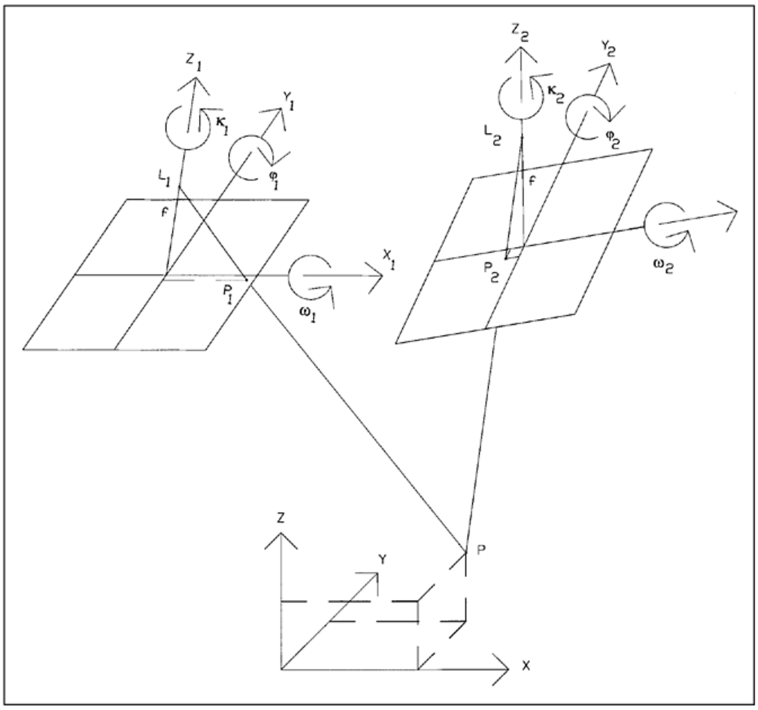
With a 12-inch focal length camera, the flying height would be 2800 feet amt. At an elevation of 175 feet, the scale would be 1 foot divided by (2800 – 175) feet, or 1 inch = 219 feet. However, at an elevation of 600 feet, the scale would be 1 foot divided by (2800 – 600) feet, or 1 inch = 183 feet. This is a relatively small variation in scale.

Normally, long-focal-length/narrow-angle lenses are better for planimetric mapping, allowing higher flying heights to be used so that views are more straight down, as looking from space. Lower flying heights with shorter-focal-length/ wide-angle lenses are better for topographic mapping where elevations are critical. The standard 6-inch lens is a compromise, and subsequent tables assume that 6-inch lenses are used.

Map Scale. Table A.4 lists the photo scale and flying height commonly used for planimetric mapping at specified map scale, assuming a standard mapping camera with a 6-inch focal length and assuming an analytical plotter or soft-copy workstation is used for the mapping. Mapping photography is normally flown with a forward overlap of 60 percent between successive photos to ensure full stereoscopic coverage, and sidelap of 30 percent between adjacent flight lines to ensure continuous coverage.

Contour Interval. Table A.5 lists the photo scale and flying height necessary for topographic mapping at specified contour intervals, assuming a standard mapping camera with a 6-inch focal length.

Figure A.14 With a strong AT solution, rays of light of features imaged on both photographs of a stereo pair will intersect and mapped coordinates will be accurate.



The most demanding requirement from these two tables controls the flying height to be used. For example, if maps with a horizontal scale of 1 inch = 50 feet (see Table A.4) are desired with a 1-foot contour interval (see Table A.5), then the photography should be flown at 2000 feet above mean terrain (instead of 2100 feet). Lower-altitude photography yields more accurate maps, both horizontally and vertically, but lower-altitude photography costs more than higher-altitude photography because high-altitude photography covers a larger area per photograph and requires fewer photographs and simpler aerial triangulation.

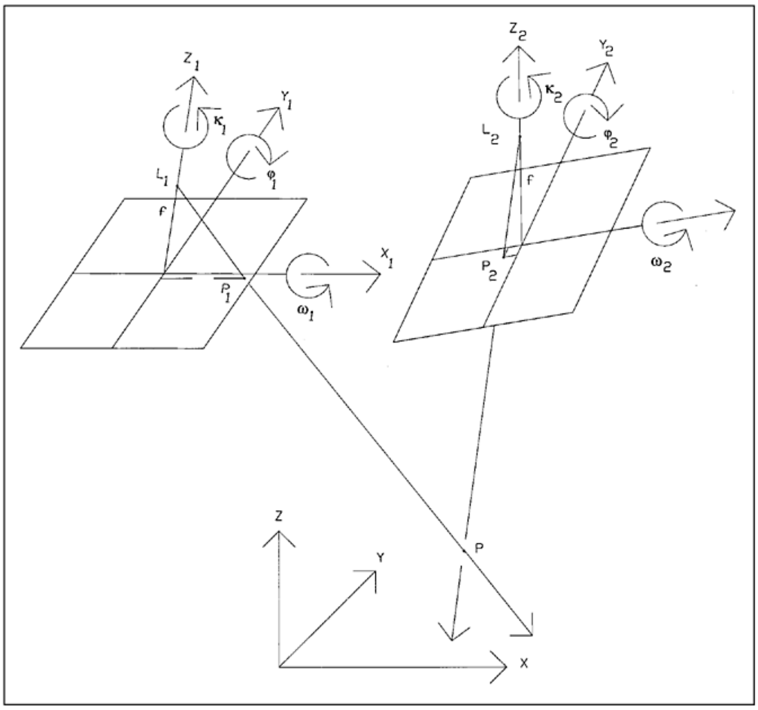
A.6.5. **Digital Topographic Data**

The introduction to this appendix included explanations of digital elevation models (DEMs), digital terrain models (DTMs), digital surface models (DSMs), mass points, breaklines, triangulated irregular networks (TINs), and contours. These are different forms of digital topographic data that are routinely produced using stereo photogrammetric techniques. Photogrammetry is so mature that procedures are well established, accuracy criteria are clear, and results are consistent, predictable, and verifiable.

A.6.6. **Digital Orthophotos**

Whereas aerial photos have perspective views of the terrain and normally include some unintentional tilt, digital orthophotos have orthographic views of the terrain (as though looking straight down on each pixel of the image from infinity) with all tilt removed. Therefore, digital orthophotos have the appearance of an aerial photograph and the metric properties of a map—with one exception. The one exception is that all features at ground level are correctly positioned, but elevated features may be displaced outward from the center of the photographs used to create the orthophotography. When a tall building, for example, appears near the center of a photograph, the orthophoto produced from that image may look like a true orthophoto without building lean; only the roof is seen. But if that same tall building appears near the edge of a 9-inch by 9-inch aerial photo, the side of the building is photographed and the roof is displaced outward from the center of the photograph and is not located directly above the building's foundation.

Figure A.15 With a weak AT solution, rays of light of features imaged on both photographs of a stereo pair will not intersect and mapped coordinates will be less accurate.



There are several different technical procedures for producing digital orthophotos, but the three points most critical for land development engineers and surveyors to understand are the following:

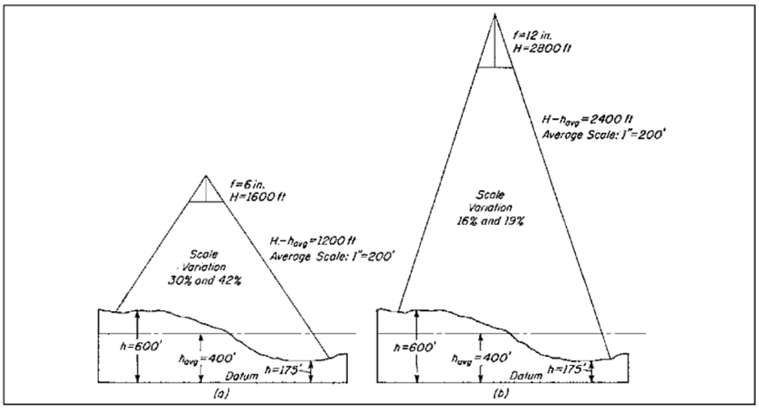
1. Digital orthophotos are relatively inexpensive and are widely considered to be ideal as the base map for overlay of CADD and/or GIS vectors, points, and polygons developed by land development engineers and surveyors.

2. Digital orthophotos contain no elevation data. They consist of image pixels with x and y coordinates only (no z values).

3. Digital orthophotos are orthographically correct only for features at ground level, whereas elevated features (tops of towers, tall trees, utility poles, rooftops) are displaced outward from the centers of the digital orthophotos, as demonstrated by the orthophoto mosaic at Figure A.17. Building lean is reduced by flying from higher altitudes using cameras with longer focal lengths and narrow-angle lenses; however, even satellite images show lean in the Washington Monument, for example.

Digital orthophotos are produced by digitizing film imagery, performing aerial triangulation so as to determine the *x-y-z* coordinates of the perspective center of each image as well as its roll, pitch, and yaw, and then projecting each correctly oriented digital image over a digital terrain model of the terrain. The DTM normally includes breaklines for the top edges of bridges so that bridges are not distorted on the imagery. In mosaicing multiple orthoimages together, care is taken to ensure that seamlines are as invisible as possible and to balance the radiometry of images so they appear to be continuous, as in Figure A.17, where the three orthophoto tiles were produced from 16 different photos. However, seamlines in water are normally obvious because of differences in sun glint at the western edge of one image where it abuts the eastern edge of an adjoining image.

Figure A.16 The average scale of a typical 9-inch by 9-inch aerial photograph depends on the focal length of the camera and the variation in terrain elevations relative to the height of the aircraft above the terrain. This figure shows two ways to acquire photographs with the same average scale but acquired with cameras having 6-inch and 12-inch focal lengths flown at two different altitudes.



A.6.7. **Oblique Aerial Imagery**

Oblique aerial imagery is deliberately acquired so as to look down at the terrain from an angle (as though looking out the window of an airliner) rather than looking straight down for mapping purposes. Several competing commercial firms specialize in oblique imagery, and they normally fly with multiple cameras, each taking photographs at set intervals of time and for which the positions (*x, y,* and *z* coordinates) and orientations (roll, pitch, and yaw) are recorded for direct georeferencing of each image. If four cameras are used, they are oriented obliquely looking northward, southward, eastward, and westward. Some have a fifth camera looking downward.

There are more aerial cameras used today in the United States for taking oblique imagery than there are aerial mapping cameras in use for taking vertical aerial photos. The reason for this is the relatively low cost and increased interpretability of oblique imagery that make them popular for public safety applications, tax assessments, and dozens of other applications.

Table A.4 Photo Scale and Flying Height for Desired Map Scale

DESIRED MAP SCALE	PHOTO SCALE	FLYING HEIGHT
1 in = 20 ft	1:1680	840 ft amt
1 in = 30 ft	1:2520	1260 ft amt
1 in = 40 ft	1:3360	1680 ft amt
1 in = 50 ft	1:4200	2100 ft amt
1 in = 60 ft	1:5040	2520 ft amt
1 in = 100 ft	1:8400	4200 ft amt
1 in = 200 ft	1:16,800	8400 ft amt

Table A.5 Photo Scale and Flying Height for Desired Contour Interval

DESIRED CONTOUR INTERVAL	PHOTO SCALE	FLYING HEIGHT
0.5 ft	1:2000	1000 ft amt
1 ft	1:4000	2000 ft amt
2 ft	1:8000	4000 ft amt
4 ft	1:16,000	8000 ft amt
5 ft	1:20,000	10,000 ft amt

Figure A.18 shows an orthophoto of a building in Los Angeles, and Figure A.19 shows an oblique image of the same building. These images were produced by the Los Angeles Region Imagery Acquisition Consortium (LAR-IAC) for which Dewberry performed independent accuracy and quality assessments. Note the improved ability to see and understand what the building really looks like from the oblique view. From the orthophoto alone (Figure A.18), the rooftop of this tall building could have been misinterpreted as many different things, including a park, a parking garage, or a short building. The oblique image (Figure A.19) allows the number of stories to be counted. The oblique image would have significantly more value to a fire department responding to an emergency at this address or to a tax appraiser needing to assess taxes.

Figures A.20 and A.21 show the value of oblique aerial imagery for postdisaster damage assessment following Hurricane Katrina. Those communities that have acquired oblique aerial imagery are much better prepared to perform accurate and rapid postdisaster damage assessments by comparing predisaster images (Figure A.20) with postdisaster images (Figure A.21). It is important to note that reasonably accurate horizontal and vertical measurements can be made on such oblique imagery. In this example, insurance claims were much better documented for rapid processing and approval of payments.

Figure A.17 This is a mosaic of three digital orthophoto tiles of the National Mall in Washington, D.C. The three photos were taken with a conventional aerial mapping camera. All features at the ground level are correctly positioned, but elevated features (e.g., towers, rooftops) are displaced outward from the centers of the individual photographs used to create the larger image mosaic. Near the southeast corner of this orthophoto mosaic, the Washington Monument is shown with its dark shadow pointing to the northwest, indicating that the photo was taken in the morning sometime around 10:30 A.M. The top of the monument is displaced to the southwest, whereas the monument base and ground features are correctly geopositioned. The center of the 9-inch by 9-inch photo that included the Washington Monument is located somewhere to the north-northeast of the monument itself, as opposed to the center of this image mosaic, which is northwest of the monument.

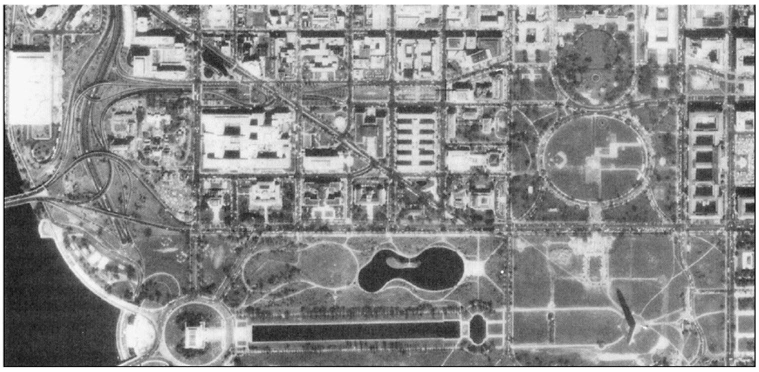


Figure A.18 Orthophoto of a building in Los Angeles.

A.6.8. **Major Points to Remember about Photogrammetry**

■ Photogrammetry is proven, mature, and well understood; results are consistent, predictable, and verifiable.

■ Stereo imagery is reused for multiple applications, that is, production of digital orthophotos, planimetric mapping, and all forms of topographic mapping. Imagery can always be used to correct errors of commission and omission at a later time.

■ Steps such as editing and finishing can always rely on a stereomodel to resolve discrepancies and correct errors to the highest accuracy the system provides.

■ Digital imagery is commonly used for quality control and to validate data from LiDAR and other sources.

■ Using stereo photogrammetry, humans manually compile breaklines and contours so that contours cross roads and streams in a manner to help humans to interpret the topography. Such contours are of cartographic quality and aesthetically pleasing, often preferred over engineering contours produced by automated techniques that are irregular and difficult to interpret.

■ Using stereo photogrammetry, humans manually compile planimetric features in 2-D or topographic features in 3-D that are very accurate.

■ The positional accuracy of digital orthophotos depends on the quality of the aerial triangulation process and the quality of the digital terrain model over which the imagery is orthorectified. Digital orthophotos have become the preferred base maps for nearly all forms of geospatial data.

■ When automated image correlation is used for generation of a digital elevation model, the resulting surface is a digital surface model rather than a DTM. It takes a human compiler to compile photogrammetric mass points on the ground only.

■ Soft-copy photogrammetry is ideal for compilation, superimposition, and display of 3-D features on top of base map imagery (digital orthophotos).

Figure A.19 Oblique aerial image of the same building.

Figure A.20 *Pre-Katrina image. (Courtesy of Pictometry)*

Figure A.21 *Post-Katrina image. (Courtesy of Pictometry)*

■ Soft-copy photogrammetry is also ideal for cut and fill calculations and for topographic modeling of future conditions (postconstruction).

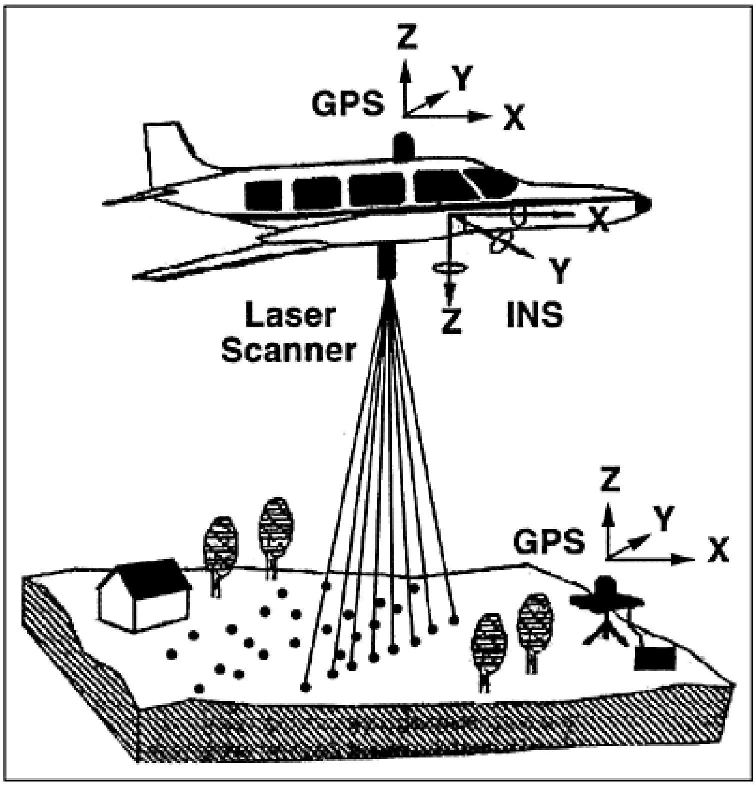
■ Photogrammetric mapping is commonly used in land development design and planning to include mapping of current and future conditions. Mapping files are routinely produced in MicroStation .dgn format, AutoCAD .dwg format, or virtually any GIS format preferred. Architectural drawings can be merged to depict future conditions in 3-D and for fly-through of a proposed development.

A.7. **LIGHT DETECTION AND RANGING (LIDAR)**

LiDAR emits thousands of laser pulses per second to accurately map features in three dimensions. Although this section pertains to aerial mapping, the same basic technology is used with terrestrial laser scanners on the ground. Both technologies are explained in detail in *Digital Elevation Model Technologies and Applications: The DEM Users Manual*.

Airborne LiDAR sensors emit up to 150,000 laser pulses per second in some form of scanning array, as shown at**Figure A.22**. The most common scanning array goes back and forth sideways, relative to the direction of flight, providing a zigzag pattern of points measured on the ground. The scan angle, flying height, and pulse repetition rate determine the nominal point spacing in the cross-flight direction, whereas the scan rate, flying height, and airspeed determine the nominal point spacing in the in-flight direction. Each laser pulse has a pulse width (typically about 1 meter in diameter) and a pulse length (equivalent to the short time lapse between the time the laser pulse was turned on and the time it was turned off again); therefore, each laser pulse actually is like a cylinder of light with diameter and length. Each laser pulse may have multiple returns from features hit at different elevations, creating a point cloud of elevation points including both treetop and rooftop elevations, as well as elevations of bare-earth mass points.

Figure A.22 LiDAR sensors survey up to 150,000 points per second, collecting high-density, high-accuracy elevation points—often several elevation points per square meter. Airborne GPS allows the x-y-z coordinates of the sensor to be known continuously. The inertial navigation system (INS) inertial measuring unit (IMU) allows the roll, pitch, and yaw of the sensor to be known continuously. The LiDAR sensor itself measures the laser scan angles and the times it takes each laser pulse to travel to the ground and reflect back to the sensor. The LiDAR point cloud data is postprocessed to classify each return as bare earth, water, vegetation, buildings, or other categories in a special laser (.LAS) file format.



Several technologies must operate correctly in order to survey high-accuracy data points:

- Airborne GPS is needed to determine the *x-y-z* coordinates of the moving sensor in the air, surveyed relative to one or more differential GPS base stations. This establishes the origin of each of the thousands of laser pulses emitted each second.

- The inertial measurement unit (IMU) directly measures the roll, pitch, and heading of the aircraft, establishing the angular orientation of the sensor about the *x, y,* and *z* axes in flight.

- The LiDAR sensor itself measures the scan angle of the laser pulses. Combined with IMU data, this establishes the angular orientation of each of the thousands of pulses emitted each second.

- The LiDAR sensor also measures the time necessary for each emitted pulse to reflect off the ground (or features thereon) and return to the sensor. Time translates into distance measured between the aircraft and the point being surveyed.

LiDAR sensors are capable of receiving multiple returns, up to five returns per pulse. For a sensor emitting 150,000 pulses per second, this means that the sensor must be capable of recording up to 150,000 × 5 = 750,000 returns per second while in flight. The first return is the top reflective surface, that is, the first thing hit by a single laser pulse; this could be a treetop, a rooftop, a ground point, or a bird in flight. When a laser pulse hits a soft target (e.g., a tree or a field of weeds), the first return represents the top or canopy of that feature. A portion of the laser light beam continues downward below the canopy and hits a tree branch, for example; this would provide a second return. Theoretically, the last return represents the bare-earth terrain, but this is often not the case. Some vegetation is so thick that no portion of the laser pulse penetrates to the ground. This is surely the case with sawgrass, mangrove, and dense forests where the canopy is so thick that a person on the ground cannot see the sky above.

A limitation of topographic LiDAR is that it normally does not penetrate water. In most cases, LiDAR pulses are absorbed by the water, so there are no returns; however, sometimes there are returns from water, but those eleva-

tions cannot be trusted. No LiDAR returns are ever thrown away, but LiDAR returns over water are classified in a water category, so they are not confused with elevations that clearly represent terrain elevations.

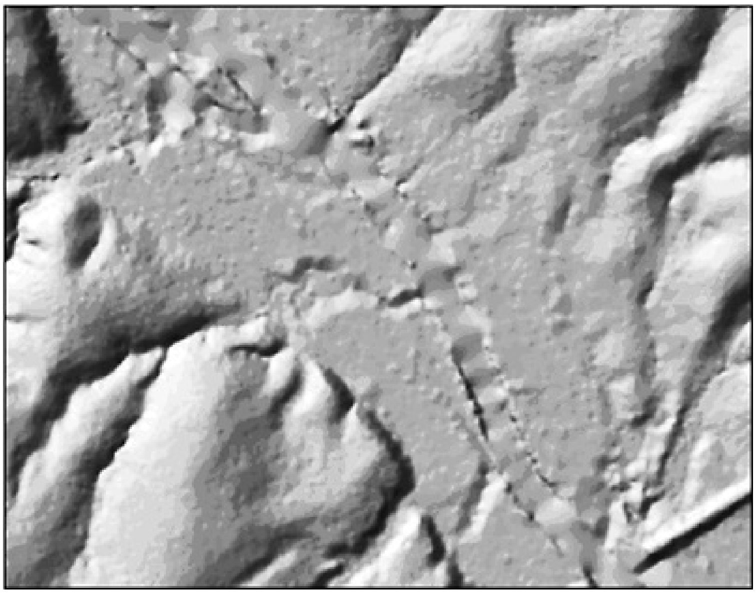
When the primary objective is to use LiDAR data for mapping the bare-earth DTM, then LiDAR data is acquired during leaf-off conditions so as to better penetrate vegetation. Whereas photogrammetry requires two (stereo) views of the bare-earth terrain from two different perspectives (a major disadvantage in vegetated terrain), LiDAR needs only a single pulse to penetrate between trees and/or to penetrate through vegetation that does not totally block light from above (a major advantage).

Postprocessing of LiDAR data is now largely automated, although manual editing is still required for quality control. The primary function of postprocessing is to classify each LiDAR return into one .LAS category, that is, a LiDAR data file format established by the ASPRS for classification of massive LiDAR point cloud datasets. As of 2007, the LAS classifications were as follows:

Class 0:	Created, never classified
Class 1:	Unclassified
Class 2:	Ground
Class 3:	Low vegetation
Class 4:	Medium vegetation
Class 5:	High vegetation
Class 6:	Buildings
Class 7:	Low point (noise)
Class 8:	Model key-point (mass point after thinning)
Class 9:	Water
Class 10:	*Reserved for ASPRS definition*
Class 11:	*Reserved for ASPRS definition*
Class 12:	Overlap points, nonground
Class 13:	*Reserved for ASPRS definition*
Class 14:	*Reserved for ASPRS definition,* e.g., bridge decks
Class 15:	*Reserved for ASPRS definition,* e.g., roads
Class 16-31	*Reserved for ASPRS definition*

LiDAR sensors are mounted in both fixed-wing aircraft and in helicopters. Fixed-wing aircraft are used for larger project areas; flight lines are straight and parallel, with occasional cross flights. Helicopters are used for small project areas, including narrow corridors that meander. In addition to mapping the terrain (as well as forest canopy for some applications), LiDAR is excellent for mapping and computing the volumes of stockpiles or for determining changes to the terrain as a result of strip mining or construction activities, for example. Because of the cost of deploying a LiDAR sensor on-site, there is normally a minimal cost involved (e.g., $25,000) even if all data can be acquired in a single day. Thus, for topographic mapping of a small construction site, for example, it is normally more cost effective to use photogrammetric mapping—so long as dense vegetation does not disallow the use of stereo photogrammetry for mapping the floor of a forest, for example.

Figure A.23a LiDAR DTM does not clearly depict the shoreline of the river, yet a digital orthophoto clearly shows such shorelines.



Whether photogrammetry or LiDAR technology is used, products are less expensive if the client can accept digital surface models instead of digital terrain models.

LiDAR data also includes intensity imagery that looks similar to digital orthophotography but instead depicts the brightness of each laser return. As indicated, LiDAR returns on water are unreliable, and it is frequently difficult to see the break line at a shoreline. Whereas orthophotos may be used to generate 2-D breaklines of shorelines (see Figure A.23*a* and *b*), lidargrammetry is now used largely to generate 3-D breaklines from LiDAR intensity imagery. Per the example in Figure A.24*a* and *b*, 3-D breaklines are required for hydroenforcement of streams to ensure that water flows downstream in hydrologic and/or hydraulic models. LiDAR "stereo mates" can be generated with base-height ratios that provide vertical exaggeration; intensity image stereo mates appear as though they are stereo photographs for compilation of 3-D breaklines and/or contours using the very same soft-copy photogrammetry tools used for compilation from stereo photographs. Although lidargrammetry is new, research has indicated that breaklines and contours produced from lidargrammetry are more accurate than breaklines and contours produced from photogrammetry, especially in terrain where vegetation obscures the stereo views required with photogrammetry.

Figure A.23b The addition of 2-D breaklines from digital orthophotos can be used to burn in the water surface at elevations lower than the terrain.

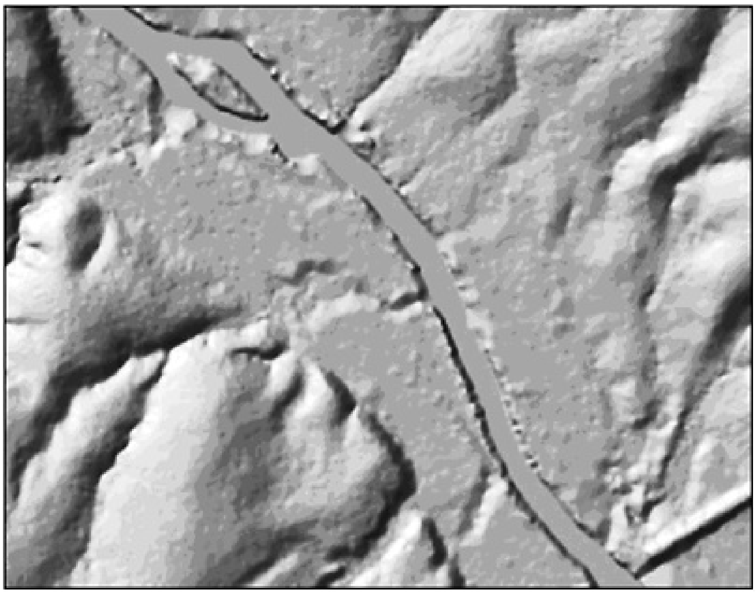


Figure A.24a High points on the shore on both sides of the river make this TIN look as though water cannot flow through this reach of the river.

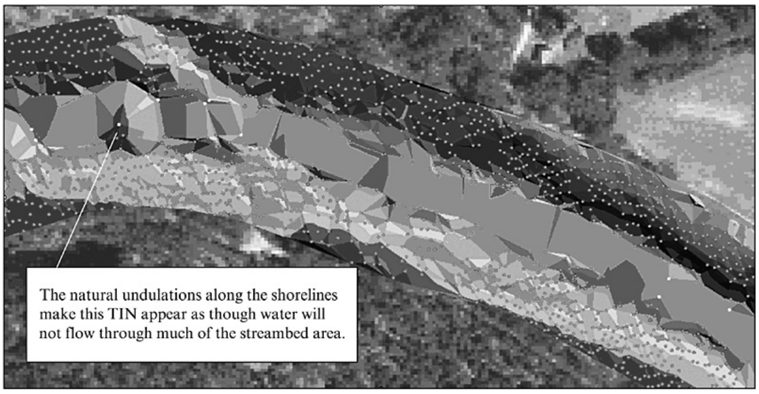
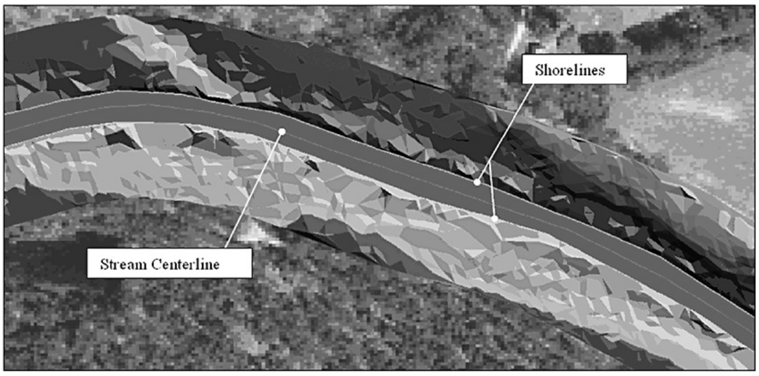


Figure A.24b The addition of 3-D breaklines from either photogrammetry or lidargrammetry are used to hydroenforce the TIN so that water continuously flows downstream.



The high-density, high-accuracy LiDAR mass points can be a mixed blessing. For example, they cause contours to appear very noisy and irregular. To overcome this limitation, it is common to delete LiDAR mass points on road pavements between edge-of-pavement breaklines. Also, when breaklines are generated for road edge of pavement, curbs, bottoms of drainage ditches, tops and bottoms of stream banks, and shorelines, for example, it is common to remove/ reclassify LiDAR points within 1 to 2 feet of those breaklines so that the advantages of clearly defined breaklines are not offset by noisy LiDAR data points nearby.

A.7.1. **Major Points to Remember about LiDAR**

■ LiDAR has an advantage over photogrammetry in generating accurate elevation datasets in dense vegetation.

■ LiDAR is ideal for generation of high-density, highaccuracy elevation datasets of large areas.

■ LiDAR is the technology of choice for generation of digital topographic datasets with accuracies equivalent to 1-foot and 2-foot contours (see **Table A.2**).

■ Although LiDAR had previously been considered weak for generation of breaklines, lidargrammetry has caused a paradigm shift in such thinking.

■ Tools are rapidly evolving for increased automation in the processing of LiDAR data, causing costs to lower significantly since 2000. This technology is evolving rapidly, and potential applications are limitless.

■ Although automation causes costs to decrease, cost competition and demands for rapid delivery of products causes deliverables from LiDAR (and photogrammetry) to be largely unseen by human eyes prior to delivery to the customer. This increases the need for independent quality assurance/quality control (QA/QC) to ensure that each client receives the quality data that it pays for.

■ Where possible, select the independent QA/QC specialist and clearly define the acceptance criteria before contracting with any vendor to produce LiDAR or photogrammetric mapping products.

A.7.1. **REFERENCES**

1. American Society for Photogrammetry and Remote Sensing. 1980. Manual of Photogrammetry, 4th ed. Bethesda, MD: American Society for Photogrammetry and Remote Sensing.
2. American Society for Photogrammetry and Remote Sensing. 1990. ASPRS Accuracy Standards for Large-Scale Maps. Photogrammetric Engineering and Remote Sensing 56(7): 1068–1070.
3. American Society for Photogrammetry and Remote Sensing. 1996. Digital Photogrammetry: An Addendum to the Manual of Photogrammetry. Bethesda, MD: American Society for Photogrammetry and Remote Sensing.
4. American Society for Photogrammetry and Remote Sensing. 2007. Digital Elevation Model Technologies and Applications: The DEM Users Manual, 2nd ed. Bethesda, MD: American Society for Photogrammetry and Remote Sensing.
5. Bureau of the Budget. 1947. National Map Accuracy Standards. Washington, DC: Office of Management and Budget.
6. Federal Geographic Data Committee. 1998a. Geospatial Positioning Accuracy Standards, Part 1: Reporting Methodology. Reston, VA: Federal Geographic Data Committee, c/o U.S. Geological Survey.
7. Federal Geographic Data Committee. 1998b. Geospatial Positioning Accuracy Standards, Part 2: Standards for Geodetic Networks. Reston, VA: Federal Geographic Data Committee, c/o U.S. Geological Survey.
8. Federal Geographic Data Committee. 1998c. Geospatial Positioning Accuracy Standards, Part 3: National Standard for Spatial Data Accuracy. Reston, VA: Federal Geographic Data Committee, c/o U.S. Geological Survey.
9. Federal Geographic Data Committee. 1999. Content Standards for Framework Land Elevation Data. Reston, VA: Federal Geographic Data Committee, c/o U.S. Geological Survey.
10. Kleusberg, Alfred. July 1992. Precise Differential Positioning and Surveying. GPS World. p. 50.
11. National Oceanic and Atmospheric Administration. 1997. Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm), Version 4.3. NOAA Technical Memorandum, NOS NGS-58. Silver Spring, MD: National Oceanic and Atmospheric Administration.