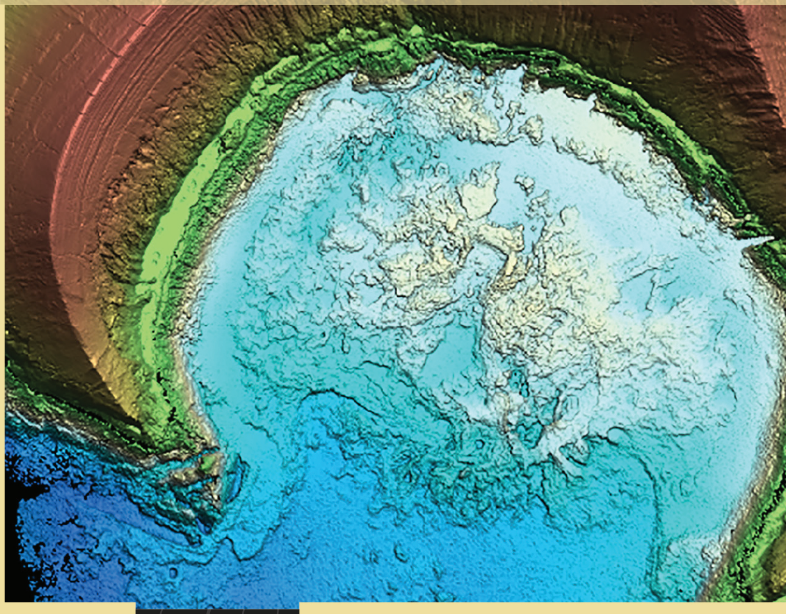
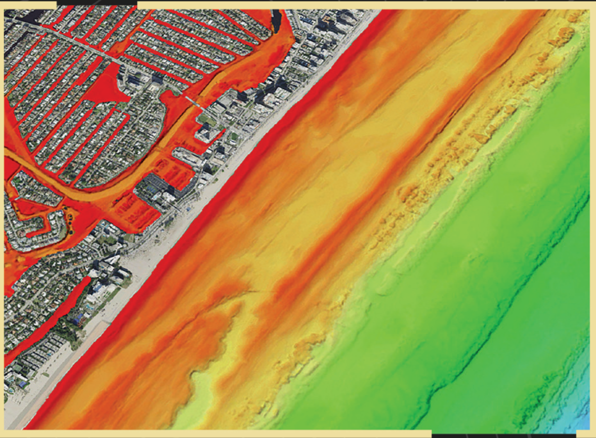
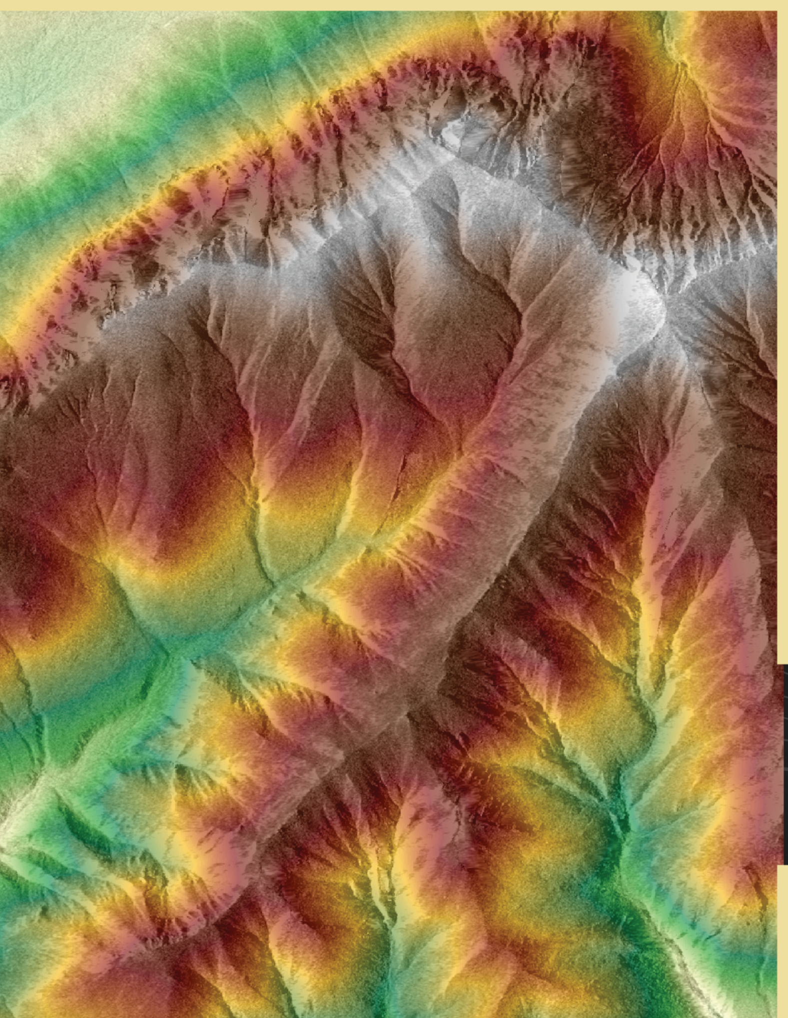
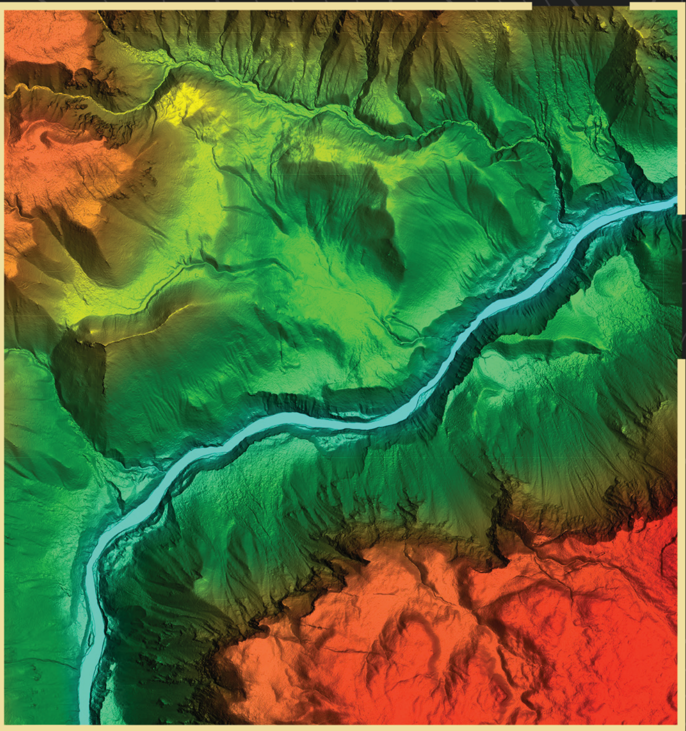
ASPRS Positional Accuracy Standards for Digital Geospatial Data

Edition 2, Version 2 (2024)









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ASPRS Positional Accuracy Standards for Digital Geospatial Data
Edition 2, Version 2 (2024)

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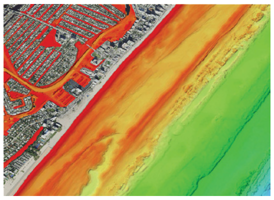
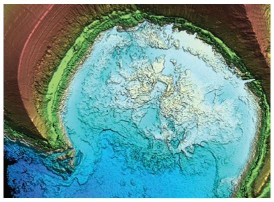
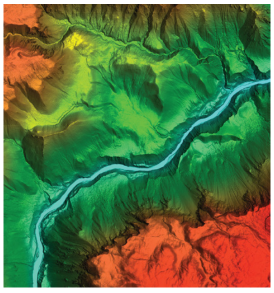


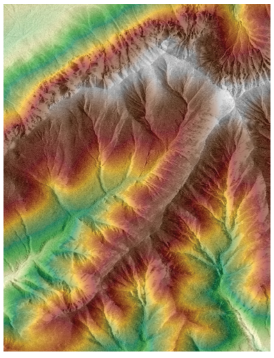
Topo-bathy Lidar Elevation Data, Fort Lauderdale, Florida, U.S.



Topo-bathy Lidar for the Commonwealth of the Northern Mariana Islands where the first islands settled by humans in remote Oceania



Topo Lidar for the last free-flowing river in the U.S., the Yellowstone River courses 700 miles across Wyoming, Montana, and North Dakota



Topo Lidar for Nenana, Alaska, U.S. where the lowest recorded temperature was -69 degrees Fahrenheit (-56 degrees Celsius)

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ASPRS Positional Accuracy Standards for Digital Geospatial Data

EDITION 2, VERSION 2 (2024)

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FOREWORD

Edition 1 of the ASPRS Positional Accuracy Standards for Digital Geospatial Data was published in November 2014. In the years since, users expressed concerns and suggested revisions based on their experience applying the Standards in real-world situations. In addition, technologies have evolved in such a way as to challenge the assumptions upon which Edition 1 was based.

In 2022, ASPRS established a formal Positional Accuracy Standards Working Group under the Standards Committee to evaluate user comments, consider technology advancements, and implement appropriate changes to the Standards. The following individuals were appointed to the Positional Accuracy Standards Working Group:

Chair: Dr. Qassim Abdullah, Vice President and Chief Scientist, Woolpert, Inc.

Members:

- Dr. Riadh Munjy, Professor of Geomatics Engineering, California State University, Fresno
- Josh Nimetz, Senior Elevation Project Lead, U.S. Geological Survey
- Michael Zoltek, National Geospatial Programs Director, GPI Geospatial, Inc.
- Colin Lee, Photogrammetrist, Minnesota Department of Transportation

The ASPRS Positional Accuracy Standards for Digital Geospatial Data are developed based on industry consensus. They are designed to be modular in nature, such that revisions can be made and additional sections added as geospatial technologies and methods evolve. Additionally, the Standards are designed to recommend best practices, methods, and guidelines for the use of emerging technologies to achieve the goals and requirements set forth in the Standards. With support from the ASPRS Technical Divisions, the primary Working Group established subordinate Working Groups to author Addenda for best practices and guidelines for photogrammetry, lidar, UAS, and field surveying. The subordinate Working Group members and contributors are credited in each Addendum.

Summary of Changes in Edition 2

Important changes adopted in Edition 2 of the Standards are as follows:

1. Eliminated references to the 95% confidence level as an accuracy measure.

   · *Reason for the change:* The 95% confidence measure of accuracy for geospatial data was introduced in the National Standard for Spatial Data Accuracy (NSSDA), published by the Federal Geographic Data Committee in 1998. This measure was carried forward in the ASPRS Guidelines for Vertical Accuracy Reporting for Lidar Data, published in 2004, as well as in Edition 1 of the ASPRS Positional Accuracy Standards for Digital Geospatial Data, published in 2014. However, Root Mean Square Error (RMSE) is also a way to express data accuracy, and it is typically reported alongside the 95% confidence level because the two are derived from the same error distribution. Due to the fact that users need to compute RMSE first in order to obtain the 95% confidence measure, the reporting of two quantities representing the same accuracy at different confidence levels has created confusion for users and data producers alike.

- *Justification for the change:* RMSE is a reliable statistical term that is sufficient to express product accuracy, and it is well understood by users. Experience has shown that the use of both RMSE and the 95% confidence level leads to confusion and misinterpretation.

2. Relaxed the accuracy requirement for ground control points and checkpoints.

- *Reason for the change:* Edition 1 called for the accuracy of ground control points to be 4 times the accuracy of the intended final product, and the accuracy of ground checkpoints to be 3 times the accuracy of the intended final product. With goals for final product accuracies approaching a few centimeters in both the horizontal and vertical, it becomes difficult, if not impossible, to use Real-Time Kinematic (RTK) methods for ground control point and checkpoint surveys, introducing a significant burden of cost for many high-accuracy projects.
- *Justification for the change:* As the demand for higher-accuracy geospatial products grows, accuracy requirements for the surveyed ground control and checkpoints set forth in Edition 1 exceed those that can be achieved in a cost-effective manner, even with high-accuracy GPS. Furthermore, today's sensors, software, and processing methods have become very precise, diminishing the errors introduced in data acquisition and processing. If best practices are followed, safety factors of 3 and 4 times the intended product accuracy are no longer needed.

3. Required the inclusion of survey checkpoint accuracy when computing the accuracy of the final product.

- *Reason for the change:* Since checkpoints will no longer need to meet the three-times-intended-product accuracy requirement (see item 2 above), the error in the checkpoints survey may no longer be ignored when reporting the final product accuracy. This is especially important, given the increasing demand for highly accurate products—which, in some cases, approach the same order of magnitude as the survey accuracy of the checkpoints. Therefore, checkpoint error should be factored into the final product accuracy assessment that is used to communicate the reliability of the resulting final products.
- *Justification for the change:* Errors in the survey checkpoints used to assess final product accuracy, although small, can no longer be neglected. As product accuracy increases, the impact of error in checkpoints on the computed product accuracy increases. When final products are used for further measurements, calculations, or decision making, the reliability of these subsequent measurements can be better estimated if the uncertainty associated with the checkpoints is factored in.

4. Removed the pass/fail requirement for Vegetated Vertical Accuracy (VVA) for lidar data.

- *Reason for the change:* Data producers and data users have reported that they are challenged in situations where Non-Vegetated Vertical Accuracy (NVA) is well within contract specifications, but VVA is not. As explained below, factors affecting VVA are not a function of the lidar system accuracy; therefore, only NVA should be used when making a pass/fail decision for the overall project. VVA should be evaluated and reported but should not be used as a criterion for acceptance.

- *Justification for the change:* Where lidar can penetrate to bare ground under trees, the accuracy of the points, as a function of system accuracy, should be comparable to lidar points in open areas. However, the accuracy and the quality of lidar-derived surface under trees is affected by:
  1) the type of vegetation where it affects the ability of lidar pulses to reach the ground,
  2) the density of lidar points reaching the ground,
  3) and the performance of the algorithms used to separate ground and above-ground points in these areas.
- Furthermore, unless the checkpoint survey in vegetated areas is performed according to the best practices and guidelines presented in Addendum II using alternate techniques, common GNSS-based survey practices are subject to many technical challenges that may impact the accuracy determination in vegetated areas. As a result, accuracy computed from the lidar-derived surface in vegetated areas is not always a valid measure for lidar system accuracy—and, therefore, for the accuracy of the terrain within. For more information about the concerns surrounding the VVA concept, please refer to Section 7.8.

5. Increased the minimum number of checkpoints required for product accuracy assessment from 20 to 30.

- *Reason for the change:* In Edition 1, a minimum of 20 checkpoints are required for testing positional accuracy of the final mapping products. This minimum number is not based on rigorous science or statistical theory; rather, it is a holdover from legacy Standards and can be traced back to the National Map Accuracy Standards published by the U.S. Bureau of the Budget in 1947.
- *Justification for the change:* The Central Limit Theorem calls for at least 30 samples to calculate statistics such as mean, standard deviation, and skew. These statistics are relied upon in positional accuracy assessments. According to The Central Limit Theorem, regardless of the distribution of the population, if the sample size is sufficiently large ($n \geq 30$), then the sample mean is approximately normally distributed, and the normal probability model can be used to quantify uncertainty when making inferences about a population based on the sample mean. Therefore, in Edition 2, a product accuracy assessment must have a minimum number of 30 checkpoints to be considered fully compliant.

6. Limited the maximum number of checkpoints for large projects to 120.

- *Reason for the change:* Since these Standards recognize the Central Limit Theorem as the basis for statistical testing, there is insufficient evidence for the need to increase the number of checkpoints indefinitely as the project area increases. The new maximum number of checkpoints is equal to four times the number called by the Central Limit Theorem.
- *Justification for the change:* According to the old guidelines, large projects require hundreds, sometimes thousands of checkpoints to assess product accuracy. Such numbers have proven to

be unrealistic for the industry, as it inflates project budget and, in some cases, hinders project executions, especially for projects taking place in remote or difficult-to-access areas.

7. Introduced a new accuracy term: “three-dimensional positional accuracy."

- *Reason for the change:* Three-dimensional models require consideration of three-dimensional accuracy, rather than separate horizontal and vertical accuracies. Edition 2 endorses the use of the following three terms:
    - Horizontal positional accuracy
    - Vertical positional accuracy
    - Three-dimensional (3D) positional accuracy
- *Justification for the change*: Three-dimensional models and digital twins are gaining acceptance in many engineering and planning applications. Many future geospatial data sets will be in true three-dimensional form; therefore, a method for assessing positional accuracy of a point or feature within the 3D model is needed to support future innovation and product specifications.

8. Added Best Practices and Guidelines Addenda for:
    - General Best Practices and Guidelines
    - Field Surveying of Ground Control Points and Checkpoints
    - Mapping with Photogrammetry
    - Mapping with Lidar
    - Mapping with Unmanned Aerial Systems (UAS)
    - Mapping with Oblique Imagery

This summarizes the most significant changes implemented in Edition 2 of the ASPRS Positional Accuracy Standards for Digital Geospatial Data. Other minor changes will be noted throughout.

Foreword to Edition 1 of 2014

The goals of American Society for Photogrammetry and Remote Sensing (ASPRS) are: to advance the science of photogrammetry and remote sensing; to educate individuals in the science of photogrammetry and remote sensing; to foster the exchange of information pertaining to the science of photogrammetry and remote sensing; to develop, place into practice, and maintain standards and ethics applicable to aspects of the science; to provide a means for the exchange of ideas among those interested in the sciences; and to encourage, publish and distribute books, periodicals, treatises, and other scholarly and practical works to further the science of photogrammetry and remote sensing.

These Standards were developed by the ASPRS Map Accuracy Standards Working Group, a joint committee under the Photogrammetric Applications Division, Primary Data Acquisition Division, and Lidar Division. The Working Group was formed for the purposes of reviewing and updating ASPRS map accuracy Standards to reflect current technologies. A subcommittee of this group, consisting of Dr. Qassim Abdullah of Woolpert, Inc., Dr. David Maune of Dewberry Consultants, Doug Smith of David C. Smith and Associates, Inc., and Hans Karl Heidemann of the U.S. Geological Survey, was responsible for drafting the document.

1. PURPOSE

The objective of Edition 2 of the ASPRS Positional Accuracy Standards for Digital Geospatial Data is to update Edition 1 of the Standards.

These Standards include positional accuracy standards for digital orthoimagery, digital planimetric data, and digital elevation data. Accuracy classes, based on RMSE values, have been revised and upgraded from the 1990 Standards to address the higher accuracies and higher spatial resolutions achievable with newer technologies. Edition 2 also introduces additional accuracy measures, such as orthoimagery seam lines, aerial triangulation accuracy, ground control point accuracy, lidar-relative swath-to-swath precision and recommended minimum Nominal Pulse Density (NPD), horizontal accuracy of elevation data, delineation of low confidence areas for vertical data, and the required spatial distribution and number of checkpoints based on project area. Edition 2 introduces major changes to Edition 1 of the Standards. The changes summarized in the Foreword were made based on the feedback received from the users of the Standards, the state of sensors technologies, and the current industry requirements.

1.1 Scope and Applicability

These Standards are intended to be broadly based, technology independent, and applicable to most common mapping applications and projects. Specifically, these Standards are to be used by geospatial data producers and data users to determine the positional accuracy requirements for final geospatial products; it does not, however, address classification accuracy for thematic maps.

New to this edition, these Standards provide best practices and guidelines recommended to meet the accuracy thresholds stated herein. Detailed testing methodologies are specified, as are key elements to be considered in data acquisition and processing for products intended to meet these Standards. However, it is ultimately the responsibility of the data producer to set forth project design parameters, processing steps, and quality control procedures to ensure all data and derived products meet specified project accuracy requirements.

1.2 Limitations

Edition 2 of these Standards addresses accuracy thresholds and testing methodologies achievable with current technology. It also addresses shortcomings in Edition 1, as indicated by users of the Standards over the decade following the first edition's publication.

Additional accuracy assessment needs identified by the Working Group but not addressed in Edition 2 include:

- Positional accuracy of linear features (as opposed to well-defined points).
- Rigorous total propagated uncertainty (TPU) error modeling.
- Robust statistics for data sets that do not meet the criteria for normally-distributed error.
- Image quality factors, such as edge definition, color balance, and contrast.
- Robust assessment of the distribution and density of ground control points and checkpoints.

These Standards are intended to be a living document which can be updated in future editions to reflect changing technologies and user needs. As stated in the Foreword, Edition 2 includes six Addenda on Best Practices and Guidelines. Subject matter experts are encouraged to develop and submit additional Addenda to ASPRS for review and publication.

To date, these Standards do not reference existing international Standards. These references could be considered as part of a future edition. Any use of trade, firm, or product names is for descriptive purposes only and does not imply endorsement by ASPRS.

1.3 Structure and Format

Primary terms and definitions, references, and requirements are stated within the main body of the Standards (Sections 1 through 7).

Detailed supporting background information and accuracy conversion examples are given in Appendices A through D:

- Appendix A provides a background summary of other standards, specifications, and guidelines that are relevant to ASPRS, but which do not satisfy current requirements for digital geospatial data.
- Appendix B provides accuracy/quality examples and overall guidelines for implementing the Standards.
- Appendix C provides guidelines for accuracy testing and reporting.
- Appendix D provides examples on computing product accuracy according to Edition 2 of these Standards.

Addenda I through VI present best practices and guidelines in the following areas of practice:

- Addendum I: General Best Practices and Guidelines

- Addendum II: Best Practices and Guidelines for Field Surveying for Ground Control Points and Checkpoints
- Addendum III: Best Practices and Guidelines for Mapping with Photogrammetry
- Addendum IV: Best Practices and Guidelines for Mapping with Lidar
- Addendum V: Best Practices and Guidelines for Mapping with Unmanned Aerial Systems (UAS)
- Addendum VI: Best Practices and Guidelines for Mapping with Oblique Imagery

2. Conformance

No conformance requirements are established for these Standards.

3. References

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National Digital Elevation Program (NDEP), 2004. *NDEP Guidelines for Digital Elevation Data*, URL: *https://publicdocuments.asprs.org/2004-NDEPGuidelines*.

National Geodetic Survey (NGS), 1997. NOAA Technical Memorandum NOS NGS-58, Version 4.3: Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm), URL: *https://www.ngs.noaa.gov/PUBS_LIB/NGS-58.html*.

National Geodetic Survey (NGS), 2008. NOAA Technical Memorandum NOS NGS-59, Version 1.5: Guidelines for Establishing GPS-Derived Orthometric Heights, URL: http://www.ngs.noaa.gov/PUBS_LIB/NGS592008069FINAL2.pdf

Informative references for additional relevant guidelines and specifications are included in Appendix A.

4. Authority

The organization responsible for preparing, maintaining, and coordinating work on these Standards is the American Society for Photogrammetry and Remote Sensing (ASPRS). The Working Group on Positional Accuracy Standards was formed under the auspices of the ASPRS Standards Committee to consider user feedback and author revisions appearing in Edition 2. For further information, contact the ASPRS Standards Committee at standardscommittee@asprs.org.

5. Terms and Definitions

- *accuracy* – The closeness of an estimated value (measured or computed, for example) to a standard or accepted (true) value of a particular quantity together with an explicit reference to the specific standard or accepted value. Not to be confused with *precision*.
    - *absolute accuracy* – A measure that accounts for all systematic and random positional errors in a data set when the data set is referenced to a known and explicitly-specified datum.
    - *product accuracy* – Actual achieved accuracy (horizontal, vertical, and three-dimensional positional accuracy) computed by statistical means for a geospatial dataset.
    - *target accuracy* – Intended accuracy (horizontal, vertical, and three-dimensional positional accuracy) of the final product.
- *accuracy class* (horizontal, vertical, and three-dimensional positional accuracy classes) – The quantification of a quality threshold in a project's scope of work. Accuracy class is expressed in this and previous editions of the standards as a function of the mapping product's RMSE.
- *bias* – A systematic error inherent in measurements, due to some deficiency in the measurement process or subsequent processing. Biases can be detected, quantified, and removed if a correct procedure is followed. Biases should be removed from a data set before accuracy assessment is performed.
- *blunder* – A mistake resulting from carelessness or negligence. Not to be confused with *error*; refer to positional error below for more information.
- *boresighting* - The process of determining the angular rotations between the body of the sensor like Lidar or camera and the body of the Inertial Measurement Unit (IMU).
- *confidence level* – The percentage of points within a data set that are estimated to meet the stated accuracy; e.g., accuracy reported at the 95% confidence level means that 95% of the positions in the data set will have an error with respect to true ground position that are less than the reported accuracy value.

- *data internal precision* (formerly, relative accuracy) – A measure of the variation of positional accuracy from point to point within a data set when evaluated across a leveled plane. In these Standards, it relates to the vertical quality of elevation data.
- *ground sample distance (GSD)* – The linear dimension of a sample pixel's footprint on the ground. In raw imagery, pixel size is not uniform and varies based on sensor orientation and terrain. The term "nominal GSD" refers to the average or approximate size of pixels in raw imagery. In orthorectified imagery, the GSD for all pixels is uniform and constant regardless of the terrain variation.
- *horizontal accuracy* – The horizontal (radial) component of positional error in a data set with respect to a horizontal datum at a specified confidence level. The horizontal accuracy is computed from the horizontal positional error along the X and Y axes using the following formula:

$$RMSE_H = \sqrt{{RMSE_X}^2 + {RMSE_Y}^2}$$

- *inertial measurement unit (IMU)* – A combination of multiple accelerometers and gyroscopes used to measure absolute spatial displacement. In geospatial sensors technologies, the IMU is mainly used to measure the sensor orientation angles roll, pitch, and yaw/heading.
- *mean error* – The average positional error in a set of values for one dimension (X, Y, or Z), obtained by adding all errors in a single dimension together and then dividing by the total number of errors for that dimension.
- *median error* – The middle error value when errors are ranked or arranged in order of magnitude. The term "middle" refers to the distance from the extremes and not the numerical value of error. For odd numbers of checkpoints, say 2k+1, the median is the (k+1)th error value. For even numbers, say 2k, the median is (k + (k+1))/2, the average of the two values closest to middle.
- *network accuracy* – The uncertainty in the coordinates of mapped points with respect to the geodetic datum at the specified confidence level. In other words, network accuracy measures how well coordinates approach an ideal, error-free datum.
- *non-vegetated vertical accuracy (NVA)* – The vertical accuracy of the elevation surface in open terrain or bare earth.
- *percentile* – A measure used in statistics which indicates the value below which a given percentage of observations in a group of observations fall. For example, the 95th percentile is the value (or score) below which 95 percent of the observations may be found. For accuracy testing, percentile calculations are based on the absolute values of the errors since it is the magnitude of the errors and not the sign that is of concern.
- *planimetric data* –The data in a planimetric map that displays features on the ground expressed in two dimensional coordinates.

- *positional error* – The difference between data set coordinate values and coordinate values from an independent source of higher accuracy for identical points. Positional error is measured along each of the three coordinate axes: X, Y, and Z. It should be noted that this is a somewhat loose usage of the term “error,” which, formally, is the difference between the measured or computed value of a quantity and its true value. Since the true values of spatial coordinates can never be known, true errors can never be known. The values referred to as “errors” throughout these Standards are more formally known as “residuals” or “residual errors”.
- *precision* – The closeness with which measurements agree with each other. Please note that, unlike RMSE, precision does not show the systematic error, or bias, if it is present in the measurements.
- *resolution* – The level of detail in an image or data, or the smallest detail a sensor can detect. In imagery, "higher resolution" means more image detail. Image resolution is usually quantified by how close alternating black and white lines can be to each other and still be visibly resolved. It is measured by lines-per-millimeter.
- *root-mean-square error (RMSE)* – The square root of the average of the set of squared differences between data set coordinate values and coordinate values from an independent source of higher accuracy for identical points.
- *skew* – A measure of the asymmetry of a probability distribution. Skewness values can be positive, zero, or negative within a data set. A skewness value near zero does not always imply that the distribution is symmetrical; however, a symmetrical distribution will always have a skew of, or close to, zero.
- *standard deviation* – A measure of spread or dispersion of a sample of errors around the sample mean error. It is a measure of precision, rather than accuracy; the standard deviation does not account for uncorrected systematic errors.
- *systematic error* – An error whose algebraic sign and, to some extent, magnitude bears a fixed relation to some condition or set of conditions. Systematic errors follow some fixed pattern and are introduced by data collection procedures, data processing, or a given datum.
- *three-dimensional positional accuracy* – The accuracy of the three-dimensional position (X, Y, and Z) of features with respect to horizontal and vertical datums as computed using the following formula:

$$RMSE_{3D} = \sqrt{{RMSE_X}^2 + {RMSE_Y}^2 + {RMSE_Z}^2}$$

- *uncertainty (of measurement)* – A parameter that characterizes the dispersion of measured values, or the range in which the “true” value most likely lies. Alternately, an estimate of the limits of the error in a measurement (where “error” is defined as the difference between the theoretically-unknowable “true” value of a parameter and its measured value). Standard uncertainty refers to uncertainty expressed as a standard deviation.
- *vegetated vertical accuracy (VVA)* – The accuracy of the elevation surface in areas where terrain is covered by vegetation.

- *vertical accuracy* – The vertical component of the positional accuracy of a data set with respect to a vertical datum, at a specified confidence level. The vertical accuracy is computed from the vertical positional error along the Z axis. Vertical accuracy is presented as $RMSE_V$. For point cloud data—whether it be from lidar, IFSAR, or photogrammetry—vertical accuracy is typically taken as a point-to-surface measurement from the checkpoint normal to the point cloud surface.

For additional terms and more comprehensive definitions, refer to the Glossary of Mapping Sciences (1994); the Manual of Airborne Topographic Lidar (2012); the Manual of Photogrammetry, 6th Edition (2013); and Digital Elevation Model Technologies and Applications: The DEM User's Manual, 3rd Edition (2018)—all published by ASPRS.

6. Symbols, Abbreviated Terms, and Notations

- ASPRS – American Society for Photogrammetry and Remote Sensing
- DEM – Digital Elevation Model
- DTM – Digital Terrain Model
- GCP – Ground Control Point
- GNSS – Global Navigation Satellite System
- GPS – Global Positioning System
- GSD – Ground Sample Distance
- IDW – Inverse Distance Weighing
- IFSAR – Interferometric Synthetic Aperture Radar
- IMU – Inertial Measurement Unit
- NGPS – Nominal Ground Point Spacing
- NMAS – National Map Accuracy Standards
- NPD – Nominal Pulse Density
- NPS – Nominal Pulse Spacing
- NSSDA – National Standard for Spatial Data Accuracy
- NVA – Non-Vegetated Vertical Accuracy
- RMS – Root Mean Square
    - RMS_{DZ} – the RMS of differences in elevations sampled at the same point location in overlapping swaths of lidar data
- RMSE – Root-Mean-Square Error
    - $RMSE_{3D}$ – the three-dimensional RMSE that represents both horizontal and vertical errors in a point position

  - $RMSE_H$ – the horizontal linear RMSE in the radial direction that includes both X- and Y-coordinate errors
  - $RMSE_V$ – the vertical linear RMSE that represents the vertical accuracy of elevation data
  - $RMSE_X$ – linear RMSE in the X direction (Easting)
  - $RMSE_Y$ – linear RMSE in the Y direction (Northing)
  - $RMSE_Z$ – linear RMSE in the Z direction (Elevation)
- TIN – Triangulated Irregular Network
- VVA – Vegetated Vertical Accuracy
- $\overline{x}$ – sample mean error
- σ – sample standard deviation
- γ_1 – sample skewness

7. SPECIFIC REQUIREMENTS

These Standards define accuracy classes based on RMSE thresholds for digital orthoimagery, digital planimetric data, and digital elevation data.

Accuracy testing is always recommended but may not be required for all data sets; specific requirements must be addressed in the project specifications. When testing is required:

- Horizontal accuracy shall be tested by comparing the planimetric coordinates of well-defined points in the data set with coordinates determined from an independent source of higher accuracy.
- Vertical accuracy shall be tested by comparing the elevations of the surface represented by the data set with elevations determined from an independent source of higher accuracy. This is done by comparing the elevations of the checkpoints with elevations interpolated from the data set at the same X, Y coordinates. See Section C.11 for detailed guidance on interpolation methods.
- Three-dimensional accuracy shall be tested by comparing the X, Y, and Z coordinates of well-defined points in the data set with X, Y, and Z coordinates determined from an independent source of higher accuracy.

All accuracies are assumed to be relative to the published datum as specified in the project specifications. Ground control accuracies and survey procedures should be established according to project requirements. Unless specified to the contrary, it is expected that all ground control points and checkpoints should follow guidelines for network accuracy as detailed in the Geospatial Positioning Accuracy Standards, Part 2: Standards for Geodetic Networks (FGDC-STD-007.2-1998), the NOAA Technical Memorandum NOS NGS 92, and Addendum II of these Standards. When local control is needed to meet specific accuracies or project needs, it must be clearly identified in both the project specifications and the metadata. When reporting accuracy, the number of significant digits in the reported accuracy values shall be equal to the number of significant digits in the delivered product coordinates.

7.1 Statistical Assessment of Accuracy

Horizontal accuracy is to be expressed as $RMSE_H$, derived from two horizontal error components, $RMSE_X$ and $RMSE_Y$ (see Section 7.3). Vertical accuracy is to be expressed as $RMSE_V$ (see Section 7.4). Three-dimensional positional accuracy is to be expressed as $RMSE_{3D}$, derived from the horizontal and vertical accuracy components, $RMSE_H$ and $RMSE_V$ (see Section 7.5). Furthermore, elevation data sets shall also be assessed for horizontal accuracy ($RMSE_H$) whenever possible (see Section 7.6).

More details on the application and calculation of these statistics can be found in Appendix D – Accuracy Statistics and Examples.

7.2 Systematic Error and Mean Error Assumptions

Except for vertical data in vegetated terrain, the assessment methods outlined in these Standards assume that the data set errors are normally distributed and that any significant systematic errors or biases have been removed. It is the responsibility of the data producer to test and verify that the data meets this requirement by evaluating all statistical parameters—including standard deviation, median, mean, and RMSE—as these may aid in the discovery and diagnosis of systematic errors. Evaluation of additional statistical measures, such as kurtosis and skew, are strongly advised.

Acceptable mean error may vary by project and should be negotiated between the data producer and the data user. As a rule, these Standards recommend that the mean error be less than 25% of the target RMSE specified for the project. Mean error greater than 25% of the target RMSE, whether identified pre-delivery or post-delivery, should be investigated to diagnose the cause. These findings should then be reported in the metadata. If further actions are taken to correct bias and reduce the mean error, these actions should also be reported in the metadata. If the data producer and data user agree to accept a mean error greater than 25% of the RMSE, this should be reported in the metadata as well.

When RMSE testing is performed, a discrepancy between the data set and a checkpoint that exceeds 3 times the target RMSE threshold in any component of the coordinate (X, Y, or Z) shall be interpreted as a blunder. The blunder should be investigated, explained, and corrected before the data set is considered to meet these Standards. Blunders may not be discarded without proper investigation. Removal of blunders should be explained and reported in the project metadata.

Systematic error or biases in data can be easily detected and eventually removed by analyzing the additional statistical terms that these Standards require data producers to generate, such as mean, median, and standard deviation. Users of these Standards are encouraged to review Appendix D for further information on these statistical terms, as well as guidance on how to utilize them when investigating the presence of systematic errors as presented in Addendum I, Section C.

7.3 Horizontal Positional Accuracy Standard for Geospatial Data

Table 7.1 defines the primary horizontal accuracy standard for digital data, including digital orthoimagery, digital planimetric data, scaled planimetric maps, and elevation data. These Standards specify Horizontal Accuracy Classes in terms of $RMSE_H$, the combined linear error along a horizontal plane in the radial direction. $RMSE_H$ is derived from $RMSE_X$ and $RMSE_Y$ according to the following formula:

$$RMSE_H = \sqrt{RMSE_X{}^2 + RMSE_Y{}^2}$$

Previous ASPRS standards used discrete, numerically-ranked accuracy classes tied to map scale (i.e., Class 1, Class 2, Class 3). Many modern applications of geospatial data call for horizontal accuracies that are not tied directly to compilation scale, resolution of the source imagery, or final pixel resolution. Therefore, these Standards allow more flexibility; they do not classify horizontal accuracy discretely, nor do they tie accuracy class to map scale.

According to these Standards, horizontal accuracy needs should be determined by project requirements, and the Horizontal Accuracy Class of a data set should be expressed as a function of $RMSE_H$. For example, a project's scope of work could call for digital orthoimagery, digital planimetric data, or scaled maps produced to meet the ASPRS Positional Accuracy Standards for 7.5-cm Horizontal Accuracy Class, meaning that the $RMSE_H$ for the resulting data set must be ≤ 7.5 cm.

In the case of digital orthoimagery mosaics, an additional criterion for the allowable mismatch at seamlines of ≤ 2* $RMSE_H$ is specified in Table 7.1. It should be understood that the term $RMSE_H$ should be computed using both $RMSE_{H_1}$ and $RMSE_{H_2}$ error components, as described in Section 7.12.

Table 7.1 Horizontal Accuracy Classes for Geospatial Data

Horizontal Accuracy Class	Absolute Accuracy	Orthoimagery Mosaic Seamline Mismatch (cm)
	$RMSE_H$ (cm)	
#-cm	≤ #	≤ 2*#

Appendix B includes examples that relate accuracy classes as defined in these Standards to equivalent classes in legacy standards. Table B.4 provides $RMSE_H$ recommendations for digital orthoimagery of various pixel sizes. Table B.4 also relates Horizontal Accuracy Class and $RMSE_H$ of digital planimetric data to legacy ASPRS and NMAS Standards. The recommended associations of $RMSE_H$ and GSD presented in Table B.4 are intended to guide users through the transition from legacy to modern standards. Such associations may change in the future as mapping technologies continue to advance and evolve. These Standards do not endorse the use of GSD, map scale, or contour interval to express product accuracy. Users of these Standards need to notice that the horizontal accuracy measure in this version refers to $RMSE_H$ while in the 2014 ASPRS Positional Accuracy Standards and the 1990 ASPRS Accuracy Standards for Large-Scale Maps, it is expressed in terms of $RMSE_X$ and $RMSE_Y$. Necessary measures must be taken to convert from one another using the following formula as outlined in the example of section B.5:

$$RMSE_H = \sqrt{RMSE_X{}^2 + RMSE_Y{}^2}$$

7.4 Vertical Positional Accuracy Standard for Elevation Data

Vertical accuracy is to be expressed as $RMSE_V$ in both vegetated and non-vegetated terrain. Vertical Accuracy Classes are defined by the associated $RMSE_V$ specified for the product. It should be understood that the term $RMSE_V$ should be computed using both $RMSE_{V_1}$ and $RMSE_{V_2}$ error components as described in Section 7.12. While the Non-Vegetated Vertical Accuracy (NVA) must meet accuracy thresholds listed in Table 7.2, the Vegetated Vertical Accuracy (VVA) has no pass/fail criteria and needs only to be tested and reported as found. If the NVA meets user specifications, VVA should be accepted at the reported accuracy level.

For projects where vegetated terrain is dominant, the data producer and the data user may agree on an acceptable threshold for the VVA. Table 7.2 provides the Vertical Accuracy Class specifications for digital elevation data, including Data Internal Precision requirements where applicable, such as in lidar acquisition. Horizontal accuracy for elevation data should also be explicitly specified and reported, as discussed in Section 7.6.

Table 7.2 Vertical Accuracy Classes for Digital Elevation Data

Vertical Accuracy Class	**Absolute Accuracy**		**Data Internal Precision (where applicable)**		
	NVA $RMSE_V$ (cm)	**VVA $RMSE_V$ (cm)**	**Within-Swath Smooth Surface Precision Max Diff (cm)**	**Swath-to-Swath Non-Vegetated RMS_{DZ} (cm)**	**Swath-to-Swath Non-Vegetated Max Diff (cm)**
#-cm	*≤ #*	*As found*	*≤ 0.60*#*	*≤ 0.80*#*	*≤ 1.60*#*

Table B.5 lists 10 typical examples of Vertical Accuracy Class, $RMSE_V$, and corresponding Data Internal Precision values based on the equations shown in Table 7.2 above. Table B.6 relates Vertical Accuracy Class and $RMSE_V$ of digital elevation data to legacy ASPRS and NMAS Standards for the same examples.

The degree to which an elevation surface accurately represents terrain is not only represented by vertical agreement at ground checkpoints; accurate representation of terrain is also a function of point spacing/density. It is possible to have a very small $RMSE_V$ relative to checkpoints, even when the surface lacks sufficient resolution to represent details present in the terrain (for more on the subject, refer to Addendum I of these Standards). Table B.7 provides recommended minimum point density and point spacing at typical Vertical Accuracy Classes.

NVA should be computed based on ground checkpoints located in traditional open (bare soil, sand, gravel, and short grass) and urban (asphalt and concrete) terrain surfaces. VVA is computed based on ground checkpoints in all types of vegetated terrain, including tall weeds, crop land, brush, and fully forested areas. VVA is exempted from pass/fail testing criteria, and only needs to be tested according to the requirements set forth in these Standards. The results should then be reported in the metadata.

7.5 Three-Dimensional Positional Accuracy Standard for Geospatial Data

Table 7.3 defines the three-dimensional accuracy standard for any three-dimensional digital data as a combination of horizontal and vertical radial error. $RMSE_{3D}$ is derived from the horizontal and vertical components of error according to the following formula:

$$RMSE_{3D} = \sqrt{{RMSE_X}^2 + {RMSE_Y}^2 + {RMSE_Z}^2}$$

or

$$RMSE_{3D} = \sqrt{{RMSE_H}^2 + {RMSE_V}^2}$$

Three-dimensional positional accuracy can be computed for any type of geospatial data, as long as the horizontal and vertical positional accuracy are assessed and reported as described in Sections 7.3 and 7.4 above. Colorized point clouds and digital twins are good candidates for three-dimensional positional accuracy assessment.

Table 7.3 Three-Dimensional Accuracy Classes for Geospatial Data

3D Accuracy Class	Absolute Accuracy
	$RMSE_{3D}$ (cm)
#-cm	≤ #

7.6 Horizontal Accuracy of Elevation Data

These Standards outline horizontal accuracy testing guidelines for elevation data created from stereo photogrammetry and lidar. For other technologies, appropriate horizontal accuracies for elevation data should be negotiated between the data producer and the data user, with specific accuracy thresholds and methods based on the technology used and the project design. Determining the horizontal accuracy of lidar data is hard to achieve in the field or from the acquired data, and it is a common practice within the industry to rely on the manufacturer’s confidence in their lidar system performance when it comes to the horizontal accuracy of lidar data. Therefore, it is not necessary for users of this Standard to require the testing and reporting of the horizontal accuracy of lidar data unless there is a valid justification to do so. For a project where the horizontal accuracy of elevation data is required to be tested and reported, the data producer assumes responsibility for establishing appropriate parameters for data acquisition and testing to verify that horizontal accuracies meet the stated project requirements. Guidelines for testing the horizontal accuracy of elevation data sets are outlined in Section C.6. Users can also use the equation provided in this section to estimate the horizontal accuracy of lidar data.

Photogrammetric elevation data: For elevation data derived using stereo photogrammetry, apply the same Horizontal Accuracy Class that would be used for planimetric data or digital orthoimagery produced from the same source, based on the same photogrammetric adjustment. Horizontal accuracies, either "produced to meet" or "tested to meet," should be reported for all photogrammetrically derived elevation data sets, expressed as $RMSE_H$.

Lidar elevation data: Horizontal error in lidar-derived elevation data is largely a function of the following and can be estimated based on related parameters:

- sensor positioning error as derived from the Global Navigation Satellite System (GNSS)
- attitude (angular orientation) error as derived from the IMU
- flying height above the mean terrain

The following equation[1] provides an estimate for the horizontal accuracy for a lidar-derived data set ($RMSE_H$), assuming positional accuracy of the GNSS; roll, pitch, and yaw/heading accuracy of the Inertial Measurement Unit (IMU); and the flying height are quantified:

$$RMSE_H = \sqrt{(GNSS\ positional\ error)^2 + \left(\frac{\tan(IMU\ roll\ or\ pitch\ error) + \tan(IMU\ heading\ error)}{1.478} * flying\ height\right)^2}$$

Where:

- flying height above mean terrain is in meters;
- GNSS positional errors are radial, in meters, and can be derived from published manufacturer specifications;
- and IMU errors are in angular units and can be derived from published manufacturer specifications.

For most lidar systems used in mapping applications, other error sources, such as laser ranging and clock timing, are small contributors to the error budget and can be considered negligible when estimating horizontal error. It is worth mentioning here that lidar beam divergence, or the spread of the pulse footprint, was not factored in the previous equation because 1) most lidar systems used by the industry today have very narrow footprints, and 2) the beam divergence is sensitive to the terrain slope and the varying footprint across the field of view of the sensor, and attempting to factor it in to the calculation would add a new level of complexity to the computations.

If the desired Horizontal Accuracy Class for the lidar data has been agreed upon by the data producer and data user, then the equation above can be rearranged to solve for the recommended flying height above mean terrain (FH):

$$FH = \frac{1.478}{\tan(IMU\ roll\ or\ pitch\ error) + \tan(IMU\ heading\ error)} \sqrt{{RMSE_H}^2 - (GNSS\ positional\ error)^2}$$

Table B.8 expresses estimates of horizontal error ($RMSE_H$) as a function of flying height using an example set of GNSS and IMU errors defined in Section B.8.

[1] The method presented here is one approach developed by Dr. Qassim Abdullah; there are other methods for estimating the horizontal accuracy of lidar data sets, which are not presented here.

7.7 Low Confidence Areas in Elevation Data

In areas of dense vegetation, it can be difficult to collect reliable elevation data. This occurs in imagery where the ground is obscured or in deep shadow, or with lidar or radar imaging where there is low signal reflectance or return. These Standards recommend that such low confidence areas be delineated by polygons and reported in the metadata. Low confidence polygons are the digital equivalent to the dashed contours referred to in legacy standards. The thoroughness of such delineation should be discussed and agreed upon by the data user and data producer according to project requirements.

Section C.8 provides specific guidelines for collecting and reporting low confidence areas in elevation data.

7.8 Vegetated Vertical Accuracy and Challenging Environment

Based on the industry experience with the poor performance of different mapping techniques under vegetation, whether it be from lidar or photogrammetric methods, these Standards do not rely on the VVA results to determine the acceptability of collected project data. Lidar performance should only be evaluated in open terrain where there is no vegetation to disturb its performance. Terrain modeling in vegetated areas is always problematic for many reasons:

1. For imagery-based mapping practices, the ground under trees is not always visible. Therefore, such areas are always tagged as "low confidence areas," as illustrated in Section 7.7.
2. Similarly, for lidar mapping practices, vegetation blocks some of the lidar pulses from reaching the ground under vegetation. Depending on the density and type of vegetation, poor modeling of the terrain is to be expected due to the compromised point cloud density.
3. The survey team usually measures the elevation of the actual ground, while the lidar point cloud measures the tops of the leaves, debris, and grass overlaying the ground. The measured difference in the elevation of the terrain under vegetation undermines the assessed accuracy, thus revealing one of the weaknesses of the VVA concept. For ideal situations, it is recommended to position surveyed points on smooth surfaces underneath vegetation with the least amount of undulation or debris lying on the surface; however, this is not practical to achieve in most cases.
4. Forest floor debris moves with wind, water runoff, and animals' disturbances to the soil. In addition to the error vegetation already introduces, it also grows in height and shape over time, which can pose serious problems, especially if the survey is not performed at the same time as the airborne survey.
5. For terrain modeling with lidar, the modeling quality is dependent upon the success of the ground filter algorithm used to segment and filter the point cloud. This can lead to algorithmic or even human errors, influencing the results.
6. Some GNSS-based surveying techniques face challenges when conducted under dense vegetation. Unless extra precautions are taken or different surveying practices or techniques are used in tandem, the results of the GNSS-based survey for the checkpoints or ground control will be compromised. Users of these Standards are advised to consult the recommendations provided in Addendum II when surveying checkpoints or control points for VVA assessment.

Removing the pass/fail criteria for the VVA puts less emphasis on the accuracy of the modeling in vegetated terrain. However, performing VVA assessment serves several additional goals for users:

- For aerial lidar survey, whether there is adequate foliage penetration obtained by the lidar system.
- Whether the ground algorithm and filter are acceptable.
- Whether the density of Lidar ground points is enough to model the vegetated terrain for a given application. Some users are only interested in the quality of the terrain modeling under vegetation. For such users, more steps should be considered to improve the quality of lidar surveying and field surveying techniques. Such users may elect to collect denser point clouds and/or use different surveying practices to ensure satisfactory results.

7.9 Accuracy Requirements for Aerial Triangulation and IMU-Based Sensor Orientation

The quality and accuracy of the aerial triangulation (if performed) and/or the GNSS/IMU-based direct georeferencing play a key role in determining the final accuracy of imagery-derived mapping products.

For photogrammetric data sets, the accuracy of aerial triangulation and/or the GNSS/IMU-based direct georeferencing must be higher than the accuracy of the derived products. The accuracy of the aerial triangulation should be on the same order as the accuracy of the ground control used for the aerial triangulation, as explained in Section 7.10 below.

For GNSS/IMU-based direct georeferencing, orientation accuracy shall be evaluated by comparing coordinates of checkpoints read from the imagery (using stereo photogrammetric measurements or other appropriate methods) to coordinates of the checkpoints as determined from higher-accuracy source data.

Aerial triangulation accuracies shall be evaluated using one of the following methods:

- Comparing coordinates of checkpoints computed in the aerial triangulation solution to coordinates of the checkpoints as determined from higher-accuracy source data. Such comparison should be reported as $RMSE_{H_{1(AT)}}$ and $RMSE_{V_{1(AT)}}$. More on computing the error's component H_1 and V_1 is provided in section 7.12.1.
- Comparing coordinates read from the imagery (using stereo photogrammetric measurements or other appropriate method) to coordinates of the checkpoints as determined from higher-accuracy source data.

For projects providing deliverables that are only required to meet horizontal accuracy (orthoimagery or two-dimensional vector data), aerial triangulation errors in Z have a smaller impact on the horizontal error budget than errors in X and Y. In such cases, the aerial triangulation requirements for $RMSE_V$ can be relaxed. For this reason, these Standards recognize two different criteria for aerial triangulation accuracy:

- Aerial triangulation designed for digital planimetric data (orthoimagery and/or map) only:
  - $RMSE_{H_{1(AT)}} \leq ½ * RMSE_{H_{(MAP)}}$
  - $RMSE_{V_{1(AT)}} \leq RMSE_{H_{(MAP)}}$

Note: The exact contribution of aerial triangulation errors in Z to the overall horizontal error budget for the end products depends on ground point location in the image and other factors. Achieving $RMSE_{V_{(AT)}}$ less than or equal to target $RMSE_H$ for the final product requires a stringent workflow to control the various source of deformations within the process caused by imagery and camera parameters and other factors that typically impact the horizontal error budget.

- Aerial triangulation designed for projects that include elevation or 3D products, in addition to digital planimetric data (orthoimagery and/or map):
  - $RMSE_{H_{1(AT)}} \leq ½ * RMSE_{H_{(MAP)}}$
  - $RMSE_{V_{1(AT)}} \leq ½ * RMSE_{V_{(DEM)}}$

In the creation of any photogrammetric product, it is strongly recommended that the results of aerial triangulation be scrutinized for accuracy. In the event aerial triangulation results do not meet the criteria stated above but do meet the RMSE requirements for the final product, attention should be shifted to the accuracy of the final products. If the final products meet target accuracies, an agreement to accept the aerial triangulation results should be made between the data producer and data user. This should then be reported in the project metadata.

Section B.1 provides examples of practical applications of aerial triangulation accuracy requirements.

7.10 Accuracy Requirements for Ground Control Used for Aerial Triangulation

The accuracy of the ground control points should be twice the target accuracy of the final products, according to the following two categories:

- Ground control for aerial triangulation designed for digital planimetric data (orthoimagery and/or map) only:
  - $RMSE_{H_{(GCP)}} \leq ½ * RMSE_{H_{(MAP)}}$
  - $RMSE_{V_{(GCP)}} \leq RMSE_{H_{(MAP)}}$
- Ground control for aerial triangulation designed for projects that include elevation or 3D products, in addition to digital planimetric data (orthoimagery and/or map):
  - $RMSE_{H_{(GCP)}} \leq ½ * RMSE_{V_{(MAP)}}$
  - $RMSE_{V_{(GCP)}} \leq ½ * RMSE_{V_{(DEM)}}$

Section B.1 provides examples of the practical application of ground control accuracy requirements for aerial triangulation.

7.11 Accuracy Requirements for Ground Control Points Used for Lidar

The accuracy of the ground control points used for lidar calibration and boresighting should be twice the target accuracy of the final products. Similarly, ground checkpoints used to assess lidar data accuracy should be twice the target accuracy of the final products.

- $RMSE_{V_{(GCP)}} \leq ½ * RMSE_{V_{(DEM)}}$

Similar guidelines can be followed for other digital data acquisition technologies, such as IFSAR.

7.12 Positional Accuracy Assessment of Geospatial Data Products

Geospatial data exchanged among users should be accompanied by metadata clearly stating positional accuracy (as defined in these or equivalent standards), as positional accuracy is an important consideration in determining the applicability of the data for an intended purpose. Mislabeled or poorly-reported positional accuracy can result in catastrophic consequences.

Product accuracy assessment requires a network of checkpoints that are well distributed throughout the project area. This network should have higher positional accuracy than the product being tested. Ideally, checkpoints should be obtained using field surveying techniques as described in Addendum II, but it is also possible to obtain checkpoints from other sources if they meet the accuracy criteria defined herein. While assessing the horizontal accuracy of an orthometric or planimetric map is straightforward, assessing vertical and three-dimensional accuracies needs to be planned carefully. Vertical accuracy is computed from the vertical position error along the Z-axis. For point cloud data—whether it be from lidar, IFSAR, or photogrammetry—vertical accuracy is typically taken as a point-to-surface measurement from the checkpoint normal to the point cloud surface. Suitable surface generation methods such as TIN or IDW can be used for this accuracy assessment. Other surface modeling methods may be used as long as the method is scientifically sound and accepted by the industry.

7.12.1 First Component of Positional Error – Product Fit to Checkpoints

For each checkpoint, the surveyed coordinates should be compared to the coordinates derived from the tested product. Then, the discrepancies between the two sets of coordinates should be computed and tabulated. The product fit to checkpoints is represented by the first component of error, $\mathrm{RMSE}_{\mathrm{H}_1}$ or $\mathrm{RMSE}_{\mathrm{V}_1}$. RMSE should be computed in each dimension from all the individual computed discrepancies between the product and the checkpoints or control points in that dimension, as stated in the following formulas:

$$RMSE_X = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(x_{i(map)} - x_{i(surveyed)})^2}$$

$$RMSE_Y = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(y_{i(map)} - y_{i(surveyed)})^2}$$

$$RMSE_Z = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(z_{i(map)} - z_{i(surveyed)})^2}$$

The first component of horizontal error is:

$$RMSE_{H_1} = \sqrt{{RMSE_X}^2 + {RMSE_Y}^2}$$

The first component of vertical error is:

$$RMSE_{V_1} = RMSE_Z$$

7.12.2 Second Component of Positional Error – Survey Control and Checkpoint Error

The second component of positional error is the error of the survey of the control points and checkpoints.[1] Because these Standards have relaxed the requirement for survey point accuracy to two times the target product accuracy, as well as the high accuracy expected from the products today, these errors can no longer be considered negligible.

The second component of positional error is represented as $\mathrm{RMSE}_{\mathrm{H}_2}$ or $\mathrm{RMSE}_{\mathrm{V}_2}$, and it is the quantity reported by the field surveyor.

7.12.3 Horizontal Positional Accuracy

To compute the horizontal accuracy of a two-dimensional product, such as a planimetric map or orthorectified image, the height component of the survey point error is ignored. We assume that X (Easting) and Y (Northing) survey point errors are equal; that is, $RMSE_{X_2} = RMSE_{Y_2}$.

Using error propagation principles for Euclidean vectors:

$$Horizontal\ Product\ Accuracy\ (RMSE_H) = \sqrt{{RMSE_{H_1}}^2 + {RMSE_{H_2}}^2}$$

7.12.4 Vertical Positional Accuracy

Vertical product accuracy is computed from the first and second components of vertical error:

$$Vertical\ Product\ Accuracy\ (RMSE_V) = \sqrt{{RMSE_{V_1}}^2 + {RMSE_{V_2}}^2}$$

7.12.5 Three-Dimensional Positional Accuracy

The three-dimensional product accuracy is computed from the vertical and horizontal product accuracy:

$$RMSE_{3D} = \sqrt{{RMSE_H}^2 + {RMSE_V}^2}$$

Table 7.4 provides examples of vertical product accuracy, assuming that the vertical survey point error reported by the surveyor is $RMSE_{V_2}$ = 2.0-cm. Additional details can be found in Section C.6.

[1] Abdullah, Q., "Rethinking Error Estimations in Geospatial Data: The Correct Way to Determine Product Accuracy", PE&RS, July 2020

Table 7.4 Computing Product Vertical Accuracy

Fit to Checkpoints $RMSE_{V_1}$ (cm)	Survey Checkpoint Accuracy $RMSE_{V_2}$ (cm)	Product Vertical Accuracy $RMSE_V$ (cm)
1.00	2.0	2.24
1.50	2.0	2.50
2.00	2.0	2.83
2.50	2.0	3.20
3.00	2.0	3.61
3.50	2.0	4.03
4.00	2.0	4.47
4.50	2.0	4.92
5.00	2.0	5.39
5.50	2.0	5.85
6.00	2.0	6.32
6.50	2.0	6.80
7.00	2.0	7.28
7.50	2.0	7.76
8.00	2.0	8.25
8.50	2.0	8.73
9.00	2.0	9.22
9.50	2.0	9.71
10.00	2.0	10.20

7.13 Checkpoint Accuracy and Placement

Pursuant to Edition 2 of the Standards, checkpoints used for product accuracy assessment shall be at least two times more accurate than the required accuracy of the geospatial product being evaluated. This shall hold true for survey checkpoints, as well as checkpoints derived from other geospatial data products. To avoid a biased accuracy assessment, a checkpoint should be located away from any ground control points used in the initial processing and data calibration.

Horizontal checkpoints shall be established at well-defined points. A well-defined point is a feature for which the horizontal position can be 1) placed with a high degree of certainty in the product being tested, and 2) measured to the required degree of accuracy with respect to the geodetic datum. Well-defined points must be easily visible or identifiable on the tested product and on the ground. In the case of orthorectified imagery, when rectifying the imagery, well-defined points shall not be selected on features that are above the elevation surface. For example, the corner of a building rooftop should not be used as a horizontal checkpoint in imagery that was orthorectified using a bare-earth DEM; however, if the imagery was orthorectified using a 3D model that includes buildings, then a point on a building rooftop may be an acceptable horizontal checkpoint.

Checkpoints used for vertical accuracy assessment shall be established at locations that minimize interpolation errors when comparing the product elevation surface to the elevations of the checkpoints. These checkpoints shall be surveyed in open terrain that is flat or in areas of gentle and uniform slope, and they should not be placed near vertical artifacts or abrupt changes in elevation (preferably three meters or more away). Checkpoints used for vertical accuracy assessment are not required to meet the above requirements for well-defined points.

7.14 Checkpoint Density and Distribution

Checkpoints for accuracy assessment should be well distributed around the project area. Considerations made for challenging circumstances—such as rugged terrain, water bodies, heavy vegetation, and inaccessibility—are acceptable if agreed upon between the data producer and the data user. In no case shall the assessment of planimetric accuracy of digital orthoimagery be based on fewer than thirty (30) checkpoints. Similarly, the assessment of the NVA or VVA of elevation data should be based on no fewer than thirty (30) checkpoints each. If either horizontal or vertical accuracy is assessed using fewer than thirty (30) checkpoints, a special reporting statement should be included, as outlined in Section 7.16.1.

A quantitative methodology for the characterization of and specification for the spatial distribution of checkpoints which accounts for land cover type and project shape does not currently exist. Until such a methodology is developed and accepted, checkpoint density and distribution must be based on empirical results and simplified area-based methods.

Appendix C provides detailed guidelines and recommendations for checkpoint density and distribution. The requirements in Appendix C may be revised in the future as quantitative methods for determining the appropriate distribution of checkpoints are developed and approved.

7.15 Data Internal Precision (Relative Accuracy) of Lidar and IFSAR Data

Data internal precision assesses the internal geometric integrity of an elevation data set, without regard to survey control or absolute coordinates. These assessments can reveal potential systematic errors related to sensor stability, quality of GNSS trajectories, ranging precision, calibration of sensor models, and/or boresight alignment. Assessment of data internal precision includes two aspects of data quality: within-swath (smooth-surface) precision, and swath-to-swath precision. As stated in Table 7.2, requirements for data internal precision are more stringent than requirements for absolute accuracy.

Wherever the following assessment methods refer to raster surfaces created from lidar data, the raster cell size should be twice the nominal NPS of the lidar point cloud. Assessment of within-swath and swath-to-swath precision should be performed from these raster surfaces, using test areas in open, uniformly-sloping terrain which contains only single-return lidar points determined to be valid surface returns. Criteria for test areas are set forth in more detail in Section C.10.

To minimize sampling bias, alternate statistical methods of testing for project-wide precision can be used to supplement the elevation difference raster tests outlined below for both within-swath and swath-to-swath precision reporting. It is recommended that precision be tested and reported on a project-wide basis. If project-wide testing is not possible with the same sampling frequency as the accuracy reporting, use a minimum of thirty (30) test locations evenly distributed across the project area.

7.15.1 Within-Swath (Smooth-Surface) Precision

Within-swath precision is usually only associated with lidar collections. This type of precision is a measure of the precision of the system when detecting flat, hard surfaces. Within-swath internal precision is an indicator of ranging precision and sensor stability. Within-swath internal precision may be evaluated in single-swath data by creating two raster elevation surfaces—one from the minimum point elevation in each raster cell, and the other from the maximum point elevation in each raster cell. The two surfaces are differenced, and the maximum difference is compared to acceptable thresholds for each accuracy class as presented in Table 7.2.

Another method used to evaluate within-swath precision is to create two raster elevation surfaces—one using points with encoded scan direction flag = 0, and the other using points with encoded scan direction flag = 1. The two surfaces are then subtracted from each other to obtain the difference. There are no recommended quantitative thresholds, but this method of assessment can be helpful in revealing systematic errors in the data stemming from a hardware malfunction or a poorly-calibrated sensor model.

7.15.2 Swath-to-Swath Precision

Swath-to-swath precision for both lidar and IFSAR collections is measured in areas of open terrain within the swath overlap.

The first method of computing swath-to-swath precision is to create a surface from each of the overlapping swaths. An elevation is extracted from each surface at a number of point sample locations, and then an elevation difference is calculated for each sample point. A root-mean-square difference, RMS_{DZ}, is then calculated from all the sample differences and compared to the threshold values presented in Table 7.2.

A second method of computing swath-to-swath precision is to create two raster elevation surfaces, one from each swath. The two surfaces are differenced, and an RMS_{DZ} calculated using sample areas that are in open terrain. This approach results in a more comprehensive assessment, and also provides the user with a visual representation of the swath-to-swath differences.

Section C.10 sets forth specific criteria for selecting checkpoint locations for swath-to-swath accuracy assessment.

7.16 Accuracy Reporting

Horizontal, vertical, and three-dimensional positional accuracies shall be assessed and formally reported according to their appropriate accuracy class, using one of the statements provided in Sections 7.16.1 and 7.16.2 of these Standards.

In addition to accuracy class, related statistical quantities should be computed and reported, including:

- Residual errors at each checkpoint
- Maximum error
- Minimum error
- Mean error
- Median error
- Standard deviation
- RMSE

Besides the above listed statistical terms, the spatial distribution of errors and the geospatial data should be evaluated to identify locations with checkpoints with high residuals (outliers). This can be achieved through a graphical representation, or a plot, of the error vectors depicted in each checkpoint. These plots help diagnose blunders, sources of systematic error, or other error patterns which may exist in the data.

Product positional accuracy is reported according to one of the following scenarios:

7.16.1 Accuracy Reporting by Data User or Consultant

This type of reporting should only be based on a set of independent checkpoints. The positional accuracy of digital orthoimagery, planimetric data, and elevation data products shall be reported in the metadata in one of the manners listed below. For projects with NVA and VVA threshold requirements, two three-dimensional positional accuracy values should be reported based on the use of NVA and VVA, respectively.

- Accuracy Testing Meets ASPRS Standard Requirements

  If testing is performed using a minimum of thirty (30) checkpoints, accuracy assessment results should be reported in the form of the following statements:

  - Reporting Horizontal Positional Accuracy

    "This data set was tested to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) for a __(cm) $RMSE_H$ Horizontal Positional Accuracy Class. The tested horizontal positional accuracy was found to be $RMSE_H$ = __(cm)".

  - Reporting Non-Vegetated Vertical Accuracy

    "This data set was tested to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) for a __(cm) $RMSE_V$ Vertical Accuracy Class. The Non-Vegetated Vertical Accuracy (NVA) was found to be $RMSE_V$ = __(cm)".

  - Reporting Vegetated Vertical Accuracy

"This data set was tested to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) for a __(cm) $RMSE_V$ Vertical Accuracy Class. The Vegetated Vertical Accuracy (VVA) was found to be $RMSE_V$ = __(cm)."

- Reporting Three-Dimensional Positional Accuracy

  "This data set was tested to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) for a ___ (cm) $RMSE_{3D}$ Three-Dimensional Positional Accuracy Class. The tested three-dimensional accuracy was found to be $RMSE_{3D}$ = ___(cm) within the NVA tested area and $RMSE_{3D}$ = ___(cm) within the VVA tested area."[1]

- Accuracy Testing Does Not Meet ASPRS Standard Requirements

  If testing is performed using fewer than thirty (30) checkpoints, accuracy assessment results should be reported in the form of the following statements:

  - Reporting Horizontal Positional Accuracy

    "This data set was tested as required by ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024). Although the Standards call for a minimum of thirty (30) checkpoints, this test was performed using ONLY __ checkpoints. This data set was produced to meet a ___(cm) $RMSE_H$ Horizontal Positional Accuracy Class. The tested horizontal positional accuracy was found to be $RMSE_H$ = ___(cm) using the reduced number of checkpoints."

  - Reporting Non-Vegetated Vertical Accuracy

    "This data set was tested as required by ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024). Although the Standards call for a minimum of thirty (30) checkpoints, this test was performed using ONLY __ checkpoints. This data set was produced to meet a ___(cm) $RMSE_V$ Vertical Positional Accuracy Class. The tested vertical positional accuracy was found to be $RMSE_V$ = ___(cm) using the reduced number of checkpoints in the NVA tested area."

  - Reporting Vegetated Vertical Accuracy

    "This data set was tested as required by ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024). Although the Standards call for a minimum of thirty (30) checkpoints, this test was performed using ONLY __ checkpoints. This data set was produced to meet a ___(cm) $RMSE_V$ Vertical Positional Accuracy Class. The tested vertical positional accuracy was found to be $RMSE_V$ = ___(cm) using the reduced number of checkpoints in the VVA tested area."

  - Reporting Three-Dimensional Positional Accuracy

[1] The Three-Dimensional Positional Accuracy in vegetated areas can be omitted from this report based on a mutual agreement between data user and data producer.

"This data set was tested as required by ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024). Although the Standards call for a minimum of thirty (30) checkpoints, this test was performed using ONLY __ checkpoints. This data set was produced to meet a ___(cm) $RMSE_{3D}$ Three-Dimensional Positional Accuracy Class. The tested three-dimensional positional accuracy was found to be $RMSE_{3D}$ = ___(cm) using the reduced number of checkpoints in the NVA tested area and $RMSE_{3D}$ = ___(cm) using the reduced number of checkpoints in the VVA tested area."

7.16.2 Accuracy Reporting by Data Producer

In most cases, data producers do not have access to independent checkpoints to assess product accuracy. If rigorous testing is not performed by the data producer due to the absence of independent checkpoints, accuracy statements should specify that the data was "produced to meet" a stated accuracy. This "produced to meet'' statement is equivalent to the "compiled to meet" statement used by prior standards when referring to cartographic maps. The "produced to meet'' statement is appropriate for data producers who employ mature technologies, and who follow best practices and guidelines through established and documented procedures during project design, data processing, and quality control, as set forth in the Addenda to these Standards. However, if enough independent checkpoints are available to the data producer to assess product accuracy, it will do no harm to report the accuracy using the statement provided in Section 7.16.1 above.

If not enough checkpoints are available, but the data producer has demonstrated that they are able to produce repeatable, reliable results and thus able to guarantee the produced-to-meet accuracy, they may report product accuracy in the form of the following statements:

- Reporting Horizontal Positional Accuracy

  "This data set was produced to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) for a __(cm) $RMSE_H$ Horizontal Positional Accuracy Class.

- Reporting Non-Vegetated Vertical Accuracy

  "This data set was produced to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) for a __(cm) $RMSE_V$ Non-Vegetated Vertical Accuracy (NVA) Class."

- Reporting Vegetated Vertical Accuracy

  "This data set was produced to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) for a __(cm) $RMSE_V$ Vegetated Vertical Accuracy (VVA) Class."

- Reporting Three-Dimensional Positional Accuracy

  "This data set was produced to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) for a ___ (cm) $RMSE_{3D}$ Three-Dimensional Positional Accuracy Class within the NVA tested area and $RMSE_{3D}$ = ___(cm) within the VVA tested area."

APPENDIX A – BACKGROUND AND JUSTIFICATIONS (INFORMATIVE)

A.1 Legacy Standards and Guidelines

Accuracy standards for geospatial data have broad applications both nationally and internationally. Besides assuring that data accuracy needs are met, they also establish quality and performance specifications for the various different geospatial products and processes, including software and data processing, datum and coordinates systems, hardware and sensor performance, and auxiliary systems.

Accuracy specifications have a much narrower focus: they determine what technical requirements and acceptance criteria a geospatial product must conform to in order to be considered acceptable for a specific, set purpose. Guidelines, on the other hand, provide recommendations for acquiring, processing, and analyzing geospatial data, and are normally intended to promote consistency and best practices in the industry.

The following is a summary of standards, specifications, and guidelines relevant to ASPRS, but which do not fully satisfy current requirements for accuracy standards for digital geospatial data:

- The *National Map Accuracy Standards* (NMAS) of 1947 established horizontal accuracy thresholds for the *Circular Map Accuracy Standard* (CMAS) as a function of map scale, and vertical accuracy thresholds for the *Vertical Map Accuracy Standard* (VMAS) as a function of contour interval—both reported at the 90% confidence level. Because NMAS accuracy thresholds are a function of the map scale and/or contour interval of a printed map, they are inappropriate for digital geospatial data where scale and contour interval are changed with a push of a button without altering the underlying horizontal and/or vertical accuracy.
- The *ASPRS 1990 Accuracy Standards for Large-Scale Maps* established horizontal and vertical accuracy thresholds in terms of RMSE values in X, Y, and Z at ground scale. However, because the RMSE thresholds for Class 1, Class 2, and Class 3 products pertain to printed maps with published map scales and contour intervals, these ASPRS Standards are similarly inappropriate for digital geospatial data.
- The *National Standard for Spatial Data Accuracy* (NSSDA), published by the Federal Geographic Data Committee (FGDC) in 1998, was developed to report accuracy of digital geospatial data at the 95% confidence level as a function of RMSE values in X, Y, and Z at ground scale, unconstrained by map scale or contour interval. The NSSDA states,

  > “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% of the time. 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% of the time. The reporting accuracy standard should be defined in metric (International System of Units, SI) units. However, accuracy will be reported in English units (inches and feet) where point coordinates or elevations are reported in English units. The NSSDA uses root-mean-square error (RMSE) to estimate positional accuracy. Accuracy reported at the 95% confidence level means that 95% of

the positions in the data set will have an error with respect to true ground position that is equal to or smaller than the reported accuracy value."

The NSSDA does not define threshold accuracy values, stating, "Agencies are encouraged to establish thresholds for their product specifications and applications and for contracting purposes." Equations for converting RMSE values in X, Y, and Z into horizontal and vertical accuracies at the 95% confidence levels are provided in Appendix 3-A of the NSSDA. The NSSDA assumes normal error distributions, with systematic errors eliminated as well as possible.

- The National Digital Elevation Program (NDEP) published the *NDEP Guidelines for Digital Elevation Data* in 2004, recognizing that lidar errors of Digital Terrain Models (DTMs) do not necessarily follow a normal distribution in vegetated terrain. The NDEP developed Fundamental Vertical Accuracy (FVA), Supplemental Vertical Accuracy (SVA) and Consolidated Vertical Accuracy (CVA). The FVA is computed in non-vegetated, open terrain only, and is based on the NSSDA's $RMSE_V$ * 1.9600 because elevation errors in open terrain do tend to follow a normal distribution, especially with a large number of checkpoints. SVA is computed in individual land cover categories, and CVA is computed in all land cover categories combined. Both SVA and CVA are based on 95^{th} percentile errors (instead of RMSE multipliers) because errors in DTMs in other land cover categories—especially vegetated/forested areas—do not necessarily follow a normal distribution. While the NDEP Guidelines do establish alternative procedures for testing and reporting the vertical accuracy of elevation data sets when errors are not normally distributed, they do not provide accuracy thresholds or quality levels.
- The *ASPRS Guidelines: Vertical Accuracy Reporting for Lidar Data*, published in 2004, essentially endorsed the NDEP Guidelines, including the NDEP Guidelines' FVA, SVA and CVA reporting models and its standards for handling elevation errors when the errors are not normally distributed. Similarly to the NDEP Guidelines, the ASPRS 2004 Guidelines do not provide accuracy thresholds or quality levels.
- Between 1998 and 2010, the Federal Emergency Management Agency (FEMA) published *Guidelines and Specifications for Flood Hazard Mapping Partners* that included $RMSE_V$ thresholds, as well as requirements for testing and reporting the vertical accuracy separately for all major land cover categories within floodplains being mapped for the National Flood Insurance Program (NFIP). In its *Procedure Memorandum No. 61 – Standards for Lidar and Other High Quality Digital Topography*, dated 27 September 2010, FEMA endorsed the *USGS Draft Lidar Base Specifications V13*, relevant to floodplain mapping in areas of highest flood risk only, with poorer accuracy and point density in areas of lesser flood risks. USGS's draft V13 specification subsequently became the *USGS Lidar Base Specification V1.0* specification, summarized below. FEMA's Guidelines and Procedures only address requirements for flood risk mapping, and do not represent universal practices for accuracy standards.
- In 2012, USGS published *Lidar Base Specification, Version 1.0*, which is based on an $RMSE_V$ of 12.5 cm in open terrain and elevation post spacing no greater than 1 to 2 meters. FVA, SVA, and CVA values are also specified. This document is not a standard, but a specification for lidar data used to populate the National Elevation Data set (NED) at $1/9^{th}$ arc-second post spacing (~3 meters) for gridded digital elevation models (DEMs).

- In 2012, USGS also published the final report of the *National Enhanced Elevation Assessment* (NEEA), which considered five Quality Levels of enhanced elevation data to satisfy nationwide requirements, with each Quality Level having different $RMSE_V$ and point density thresholds. With support from the National Geospatial Advisory Committee (NGAC), USGS subsequently developed its new 3D Elevation Program (3DEP) based on lidar Quality Level 2 data with 1' equivalent contour accuracy ($RMSE_Z < 10$ cm) and point density of 2 points per square meter for all states except Alaska—for Alaska, IFSAR Quality Level 5 data is specified, meaning the $RMSE_V$ must be between 1 to 2 meters, and must have 5 meter post spacing. The 3DEP lidar data is expected to be high resolution and capable of supporting DEMs at 1 meter resolution. The 3DEP Quality Level 2 and Quality Level 5 products are expected to become industry standards for digital elevation data, effectively replacing the older elevation data from the USGS' National Elevation Dataset.
- The latest *USGS Lidar Base Specification, Version 1.2* was published in 2014 to accommodate lidar Quality Levels 0, 1, 2 and 3.
- In this version of the Standards, the accuracy measure of 95% confidence level is removed in favor of only reporting the RMSE values, due to the confusion it creates for both data producers and users of the Standards. However, when 95% confidence interval reporting is required, readers should refer to Section B.7.

A.2 A New Standard for a New Era

The current Standards were developed in response to the pressing need within the GIS and mapping community for new standards that embrace the digital nature of current geospatial technologies. The following are some of the justifications for the development of the new Standards:

- Legacy map accuracy standards, such as the ASPRS 1990 Standards and the NMAS of 1947, are outdated. Many of the data acquisition and mapping technologies that these standards were based on are no longer used. More recent advances in mapping technologies can now produce better quality and higher accuracy geospatial products and maps. New standards are needed to reflect these advances.
- Legacy map accuracy standards were designed to deal with plotted or drawn maps as the only mediums with which to represent geospatial data. The concept of hardcopy map scale dominated the mapping industry for decades. Digital mapping products need different measures (besides scale) that are suitable for the digital medium that users now utilize.
- Within the past two decades (during the transition period between the hardcopy and softcopy mapping environments), most standard measures for relating GSD and map scale to the final mapping accuracy were inherited from photogrammetric practices using scanned film. With advances in technology and in our knowledge of mapping processes and mathematical modeling, new mapping processes and methodologies have become much more sophisticated. Mapping accuracy can no longer be associated with the camera geometry and flying altitude alone; many other factors now influence the accuracy of geospatial mapping products. Such factors include: the quality of camera calibration parameters, the quality and size of the Charged Coupled Device (CCD) used in the digital camera CCD array, the amount of imagery overlap, the

quality of parallax determination or photo measurements, the quality of the GPS signal, the quality and density of ground control, the quality of the aerial triangulation solution, the capability of the processing software to handle GPS drift and shift as well as camera self-calibration, and the digital terrain model used for the production of orthoimagery. These factors can vary widely from project to project, depending on the sensor and specific methodologies used. For these reasons, existing accuracy measures based on map scale, film scale, GSD, C-factor, and scanning resolution no longer apply to the vast majority of current geospatial mapping practices.

- Elevation products from new technologies and active sensors such as lidar and IFSAR are not considered in legacy mapping standards. New accuracy standards are needed to address elevation products derived from these technologies.

A.2.1 Mapping Practices During the Film-Based Era

In the early history of photogrammetric mapping, film was the only medium capable of recording an aerial photographic session. During that period, film scale, film-to-map enlargement ratio, and C-factor were used to define final map scale and map accuracy. A film-to-map enlargement ratio value of 6 and a C-factor value of 1800 to 2000 were widely accepted and used. C-factor was used to determine the flying height based on the desired contour interval via the following formula:

$$C\text{-}factor = \frac{flying\ height}{contour\ interval}$$

Values in Table A.1 were historically utilized by the mapping community for photogrammetric mapping from film:

Table A.1 Common Photography Scales using Camera with 9” Film Format and 6” Lens

Film Scale	1” = 300’	1” = 600’	1” = 1200’	1” = 2400’	1” = 3333’
	1:3,600	1:7,200	1:14,400	1:28,800	1:40,000
Flying Height	1,800’ / 550 m	3,600’ / 1,100 m	7,200’ / 2,200 m	14,400’ / 4,400 m	20,000’ / 6,100 m
Map Scale	1” = 50’	1” = 100’	1” = 200’	1” = 400’	1” = 1000’
	1:600	1:1,200	1:2,400	1:4,800	1:12,000

A.2.2 Mapping Practices During the Softcopy Photogrammetry Era

When the softcopy photogrammetric mapping approach was first introduced to the mapping industry in the early 1990s, large format film scanners were used to convert aerial film to digital imagery. The mapping community needed guidelines for relating the scanning resolution of the film to the supported map scale and contour interval used by legacy standards to specify map accuracies. Table A.2 relates the resulting GSD of the scanned film and the supported map scale and contour interval derived from film-based cameras at different flying altitudes. Table A.2 assumes a scan resolution of 21 microns, as that was in common use for many years. The values in Table A.2 are derived based on the commonly used film-to-map enlargement ratio of 6 and a C-factor of 1800. Such values were endorsed and widely used by both map users and data producers during and after the transition period from film to the softcopy environment.

Table A.2 Relationship between Film Scale and Derived Map Scale

	Common Photography Scales (with 9" film format camera and 6" lens)			
Photo Scale	1" = 300'	1" = 600'	1" = 1200'	1" = 2400'
	1:3,600	1:7,200	1:14,400	1:28,800
Flying Height	1,800' / 550 m	3,600' / 1,100 m	7,200' / 2,200 m	14,400' / 4,400 m
Approximate Ground Sample Distance (GSD) of Scan	0.25' / 7.5 cm	0.50' / 0.15 m	1.0' / 0.3 m	2.0' / 0.6 m
	Supported Map/Orthoimagery Scales and Contour Intervals			
GSD	3" / 7.5 cm	6" / 15 cm	1.0' / 30 cm	2.0' / 60 cm
C.I.	1.0' / 30 cm	2.0' / 60 cm	4' / 1.2 m	8' / 2.4 m
Map Scale	1" = 50'	1" = 100'	1" = 200'	1" = 400'
	1:600	1:1,200	1:2,400	1:4,800

A.2.3 Mapping Practices During the Digital Sensors Photogrammetry Era

Ever since it was first introduced to the mapping community in 2000, the digital large format metric mapping camera has become the main aerial imagery acquisition system utilized for geospatial mapping. The latest generation of digital metric mapping cameras have enhanced optics quality, extended radiometric resolution through a higher dynamic range, finer CCD resolution, more durable body construction, and more precise electronics than their predecessors. These new camera technologies, coupled with advances in airborne GPS and mathematical modeling performed by modern photogrammetric processing software, make it possible to extend the limits on the flying altitude and still achieve high quality mapping products of equal or greater accuracy than what could be achieved with older technologies.

Many of the rules that have influenced photogrammetric practices for the last six or seven decades (such as those outlined in Sections A.2.1 and A.2.2 above) are based on the capabilities of outdated technologies and techniques. For instance, standard legacy guidelines, like the film-to-map enlargement ratio value of 6 and C-factors between 1800 to 2000, are based on the limitations of optical-mechanical photogrammetric plotters and aerial film resolution. These legacy rules no longer apply to mapping processes utilizing digital mapping cameras and current technologies.

Unfortunately, in the absence of clear, updated standards, outdated practices and guidelines intended for older technologies with very different limitations are commonly misapplied to newer technologies. Most users and data producers still utilize the figures given in Table A.2 for associating the imagery GSD to a supported map scale and associated accuracy, even though these associations are based on scanned film and do not apply to current digital sensors. New relationships between imagery GSD and product accuracy are needed to account for the full range of factors that influence the accuracy of mapping products derived from digital sensors.

APPENDIX B – DATA ACCURACY AND QUALITY EXAMPLES (NORMATIVE)

B.1 Aerial Triangulation and Ground Control Accuracy Examples

Sections 7.9 and 7.10 describe the accuracy requirements for aerial triangulation, IMU, and ground control points relative to product accuracies. These requirements differ depending on whether the products include elevation data. Tables B.1 and B.2 provide an example of how these requirements can be applied to a typical product with a horizontal accuracy of $RMSE_H$ equal to 50 cm.

Table B.1 Aerial Triangulation and Ground Control Accuracy Requirements – For Orthoimagery and/or Planimetric Data Only

Product Accuracy ($RMSE_H$) (cm)	AT Accuracy		Ground Control Accuracy	
	$RMSE_H$ (cm)	$RMSE_V$ (cm)	$RMSE_H$ (cm)	$RMSE_V$ (cm)
50	*25*	*50*	*25*	*50*

Table B.2 Aerial Triangulation and Ground Control Accuracy Requirements – For Orthoimagery and/or Planimetric Data and Elevation Data

Product Accuracy ($RMSE_H$) (cm)	AT Accuracy		Ground Control Accuracy	
	$RMSE_H$ (cm)	$RMSE_V$ (cm)	$RMSE_H$ (cm)	$RMSE_V$ (cm)
50	*25*	*25*	*25*	*25*

B.2 Digital Orthoimagery Horizontal Accuracy Classes

This Standard does not associate product accuracy with the GSD of the source imagery, pixel size of the orthoimagery, or map scale for scaled maps.

The relationship between the recommended $RMSE_H$ accuracy class and the orthoimagery pixel size varies depending on the imaging sensor characteristics and the specific mapping processes used. The appropriate Horizontal Accuracy Class must be negotiated and agreed upon between the end user and the data producer, based on specific project needs and design criteria. This section provides some general guidance to assist in making these decisions.

Table B.3 presents examples of 24 Horizontal Accuracy Classes and their associated orthoimagery quality criteria according to the requirements outlined in Section 7.3.

Table B.3 Common Horizontal Accuracy Classes According to the New Standard[1]

Horizontal Accuracy Class	$RMSE_H$ (cm)	Orthoimage Mosaic Seamline Maximum Mismatch (cm)
0.6	*0.6*	*1.3*
1.3	*1.3*	*2.5*
2.5	*2.5*	*5.0*
5.0	*5.0*	*10.0*
7.5	*7.5*	*15.0*
10.0	*10.0*	*20.0*
12.5	*12.5*	*25.0*
15.0	*15.0*	*30.0*
17.5	*17.5*	*35.0*
20.0	*20.0*	*40.0*
22.5	*22.5*	*45.0*
25.0	*25.0*	*50.0*
27.5	*27.5*	*55.0*
30.0	*30.0*	*60.0*
45.0	*45.0*	*90.0*
60.0	*60.0*	*120.0*
75.0	*75.0*	*150.0*
100.0	*100.0*	*200.0*
150.0	*150.0*	*300.0*
200.0	*200.0*	*400.0*
250.0	*250.0*	*500.0*
300.0	*300.0*	*600.0*
500.0	*500.0*	*1000.0*
1000.0	*1000.0*	*2000.0*

[1] For Tables B.3 through B.8, values were rounded to the nearest mm after full calculations were performed with all decimal places.

Achieving the highest level of accuracy requires specialized considerations according to sensor type, ground control density, ground control accuracies, and overall project design. In many cases, these considerations may result in unrealistic or unreasonable costs. As such, the highest achievable accuracies may not be appropriate for all projects. Many geospatial mapping projects require high-resolution and high-quality imagery, but do not require the highest level of positional accuracy. This is particularly true for map updating or similar projects where the intent is to upgrade the image resolution, but still leverage existing elevation model data and ground control data that may have been originally developed according to a lower accuracy standard.

B.3 Digital Planimetric Data Horizontal Accuracy Classes

Table B.4 presents 24 common Horizontal Accuracy Classes for digital planimetric data, approximate GSD of source imagery for high-accuracy planimetric data, and equivalent map scales per legacy NMAS and ASPRS 1990 Accuracy Standards. In Table B.4, the values for the approximate GSD of source imagery only apply to imagery derived from common large- and medium-format metric cameras. **The range of the approximate GSD of source imagery is only provided as a general reference based on the current state of sensor technologies and mapping practices, and it should not be used to reference product accuracy.** Different ranges may be considered in the future depending on technological advances and mapping practices.

Table B.4 Horizontal Accuracy/Quality Examples for High Accuracy Digital Planimetric Data

ASPRS Edition 2, Version 2 (2024)			Equivalent to Map Scale in		Equivalent to Map Scale in NMAS
Horizontal Accuracy Class	**$RMSE_H$ (cm)**	**Approximate GSD of Source Imagery (cm)**	**ASPRS 1990 Class 1**	**ASPRS 1990 Class 2**	
0.63	*0.63*	*0.31 to 0.63*	*1:25*	*1:12.5*	*1:16*
1.25	*1.25*	*0.63 to 1.25*	*1:50*	*1:25*	*1:32*
2.5	*2.5*	*1.25 to 2.5*	*1:100*	*1:50*	*1:63*
5.0	*5.0*	*2.5 to 5.0*	*1:200*	*1:100*	*1:127*
7.5	*7.5*	*3.8 to 7.5*	*1:300*	*1:150*	*1:190*
10.0	*10.0*	*5.0 to 10.0*	*1:400*	*1:200*	*1:253*
12.5	*12.5*	*6.3 to 12.5*	*1:500*	*1:250*	*1:317*
15.0	*15.0*	*7.5 to 15.0*	*1:600*	*1:300*	*1:380*
17.5	*17.5*	*8.8 to 17.5*	*1:700*	*1:350*	*1:444*
20.0	*20.0*	*10.0 to 20.0*	*1:800*	*1:400*	*1:507*
22.5	*22.5*	*11.3 to 22.5*	*1:900*	*1:450*	*1:570*
25.0	*25.0*	*12.5 to 25.0*	*1:1000*	*1:500*	*1:634*
27.5	*27.5*	*13.8 to 27.5*	*1:1100*	*1:550*	*1:697*
30.0	*30.0*	*15.0 to 30.0*	*1:1200*	*1:600*	*1:760*
45.0	*45.0*	*22.5 to 45.0*	*1:1800*	*1:900*	*1:1141*
60.0	*60.0*	*30.0 to 60.0*	*1:2400*	*1:1200*	*1:1521*
75.0	*75.0*	*37.5 to 75.0*	*1:3000*	*1:1500*	*1:1901*
100.0	*100.0*	*50.0 to 100.0*	*1:4000*	*1:2000*	*1:2535*
150.0	*150.0*	*75.0 to 150.0*	*1:6000*	*1:3000*	*1:3802*
200.0	*200.0*	*100.0 to 200.0*	*1:8000*	*1:4000*	*1:5069*
250.0	*250.0*	*125.0 to 250.0*	*1:10000*	*1:5000*	*1:6337*
300.0	*300.0*	*150.0 to 300.0*	*1:12000*	*1:6000*	*1:7604*
500.0	*500.0*	*250.0 to 500.0*	*1:20000*	*1:10000*	*1:21122*
1000.0	*1000.0*	*500.0 to 1000.0*	*1:40000*	*1:20000*	*1:42244*

B.4 Digital Elevation Data Vertical Accuracy Classes

Table B.5 provides vertical accuracy examples and other quality criteria for 10 common Vertical Accuracy Classes. Table B.6 compares the 10 Vertical Accuracy Classes with contour intervals from legacy ASPRS 1990 and NMAS 1947 Standards. Table B.7 provides 10 Vertical Accuracy Classes with the recommended lidar point density suitable for each of them.

Table B.5 Vertical Accuracy/Quality Examples for Digital Elevation Data

Vertical Accuracy Class	Absolute Accuracy		Data Internal Precision (where applicable)		
	NVA $RMSE_V$ (cm)	VVA $RMSE_V$ (cm)	Within-Swath Smooth Surface Precision Max Diff (cm)	Swath-to-Swath Non-Vegetated RMS_{DZ} (cm)	Swath-to-Swath Non-Vegetated Max Diff (cm)
1-cm	*≤ 1.0*	*As found*	*≤ 0.6*	*≤ 0.8*	*≤ 1.6*
2.5-cm	*≤ 2.5*	*As found*	*≤ 1.5*	*≤ 2.0*	*≤ 4.0*
5-cm	*≤ 5.0*	*As found*	*≤ 3.0*	*≤ 4.0*	*≤ 8.0*
10-cm	*≤ 10.0*	*As found*	*≤ 6.0*	*≤ 8.0*	*≤ 16.0*
15-cm	*≤ 15.0*	*As found*	*≤ 9.0*	*≤ 12.0*	*≤ 24.0*
20-cm	*≤ 20.0*	*As found*	*≤ 12.0*	*≤ 16.0*	*≤ 32.0*
33.3-cm	*≤ 33.3*	*As found*	*≤ 20.0*	*≤ 26.7*	*≤ 53.3*
66.7-cm	*≤ 66.7*	*As found*	*≤ 40.0*	*≤ 53.3*	*≤ 106.7*
100-cm	*≤ 100.0*	*As found*	*≤ 60.0*	*≤ 80.0*	*≤ 160.0*
333.3-cm	*≤ 333.3*	*As found*	*≤ 200.0*	*≤ 266.7*	*≤ 533.3*

Table B.6 Vertical Accuracy of the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) Compared to Legacy Standards

Vertical Accuracy Class	NVA $RMSE_V$ (cm)	Equivalent Class 1 Contour Interval per ASPRS 1990 (cm)	Equivalent Class 2 Contour Interval per ASPRS 1990 (cm)	Equivalent Contour Interval per NMAS (cm)
1-cm	*1.0*	*3.0*	*1.5*	*3.29*
2.5-cm	*2.5*	*7.5*	*3.8*	*8.22*
5-cm	*5.0*	*15.0*	*7.5*	*16.45*
10-cm	*10.0*	*30.0*	*15.0*	*32.90*
15-cm	*15.0*	*45.0*	*22.5*	*49.35*
20-cm	*20.0*	*60.0*	*30.0*	*65.80*
33.3-cm	*33.3*	*99.9*	*50.0*	*109.55*
66.7-cm	*66.7*	*200.1*	*100.1*	*219.43*
100-cm	*100.0*	*300.0*	*150.0*	*328.98*
333.3-cm	*333.3*	*999.9*	*500.0*	*1096.49*

Table B.7 Examples of Vertical Accuracy and Recommended Lidar Point Density for Digital Elevation Data according to the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024)

Vertical Accuracy Class	NVA $RMSE_V$ (cm)	Recommended Minimum NPD[1] (pls/m^2)	Recommended Maximum NPS[5] (m)
1-cm	*1.0*	*≥ 20*	*≤ 0.22*
2.5-cm	*2.5*	*16*	*0.25*
5-cm	*5.0*	*8*	*0.35*
10-cm	*10.0*	*2*	*0.71*
15-cm	*15.0*	*1*	*1.0*
20-cm	*20.0*	*0.5*	*1.4*
33.3-cm	*33.3*	*0.25*	*2.0*
66.7-cm	*66.7*	*0.1*	*3.2*
100-cm	*100.0*	*0.05*	*4.5*
333.3-cm	*333.3*	*0.01*	*10.0*

B.5 Relating ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) Values to Legacy ASPRS 1990 Accuracy Values

In this section, examples are provided for users who wish to compare these Standards to the legacy ASPRS 1990 Accuracy Standards for Large-Scale Maps. A major advantage of these Standards is that accuracy statements are based on RMSE at ground scale. The legacy standards refer to RMSE but define Class 1 as higher accuracy and Classes 2 and 3 as lower accuracy, while these Standards refer to the map accuracy by the value of RMSE without defining discrete numbered classes. The following examples illustrate the procedures users can follow to relate horizontal and vertical accuracy values between these Standards and the legacy ASPRS 1990 Accuracy Standards for Large-Scale Maps.

[1] Nominal Pulse Density (NPD) and Nominal Pulse Spacing (NPS) are geometrically inverse methods to measure the pulse density or spacing of a lidar collection. NPD is a ratio of the number of points to the area in which they are contained, and is typically expressed as pulses per square meter (ppsm or pts/m2). NPS is a linear measure of the typical distance between points, and is most often expressed in meters. Although either expression can be used for any data set, NPD is usually used for lidar collections with NPS < 1, and NPS is used for those with NPS ≥ 1. Both measures are based on all first- or last-return lidar point data, as these return types each reflect the number of pulses. Conversion between NPD and NPS is accomplished using the equation $\mathrm{NPS}=1/\mathrm{NPD}$ or $\mathrm{NPD}=1/\mathrm{NPS}$. Although typical point densities are listed for specified vertical accuracies, users may select higher or lower point densities according to project requirements and complexity of surfaces to be modeled.

Example 1: Relating the Horizontal Accuracy of a Map or Orthorectified Image calculated with the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) to the Legacy ASPRS Map Standards of 1990

Given a map or orthoimagery with an accuracy of $RMSE_H$ = 15 cm according to the 2024 Standards, compute the equivalent accuracy and map scale according to the legacy 1990 Standards.

Solution:

1. According to the legacy 1990 Standards, horizontal accuracy is represented by $RMSE_X$ or $RMSE_Y$. If we assume that $RMSE_X = RMSE_Y$, then:

   $$RMSE_H = \sqrt{RMSE_X{}^2 + RMSE_Y{}^2}$$

   or

   $$RMSE_X \text{ or } RMSE_Y = RMSE_H / 1.414 = 10.61 \text{ cm}$$

2. To find the equivalent map scale according to the legacy 1990 Standards, follow the following steps:

   a. ASPRS legacy standard of 1990 express first class accuracy as:

   Class 1 accuracy (RMSE) = Map Scale Factor / 100, where the verbal map scale is represented as 1" = X', therefore:

   Map Scale Factor (inch) = Accuracy (RMSE) x 100, or

   Map Scale Factor (cm) = Accuracy (RMSE) x 100 / 2.54 = Accuracy (RMSE) x 40

   (to simplify the computations of the multiplication constant, the inch is assumed to be equal to 2.5 cm instead of 2.54 cm)

   b. Multiply the $RMSE_X$ or $RMSE_Y$ value in centimeters by 40 to compute the map scale factor (MSF) for a Class 1 map:

   MSF = 10.61 cm * 40 = 424

   c. The map scale according to the legacy 1990 Standards are:

   Map Scale = 1: MSF or 1:424 Class 1

   The accuracy value of $RMSE_H$ = 15-cm is also equivalent to Class 2 accuracy for a map with a scale of 1:212.

Example 2: Relating the Vertical Accuracy of an Elevation Data Set calculated with the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) to the Legacy ASPRS Map Standards of 1990

Given an elevation data set with a vertical accuracy of $RMSE_V$ = 10-cm according to Edition 2 of the Standards, compute the equivalent contour interval according to the legacy 1990 Standards.

Solution:

The legacy 1990 ASPRS Standards state:

> *"The limiting RMS error in elevation is set by the Standard at one-third the indicated contour interval for well-defined points only. Spot heights shall be shown on the map within a limiting RMS error of one-sixth of the contour interval."*

Because both standards utilize the same RMSE measure to express vertical accuracy, then the accuracy of the elevation data set according to the legacy 1990 Standards are:

$RMSE_V = 10$ cm

Using the legacy 1990 Standards' accuracy measure of $RMSE_V = 1/3 * $ contour interval (CI):

$CI = 3 * RMSE_V = 3 * 10$ cm $= 30$ cm for Class 1, or

CI = 15 cm for Class 2

If the user is interested in evaluating the spot height requirement according to the legacy 1990 Standards, the accuracy for spot heights is required to be twice the accuracy of the contours (one-sixth versus one-third for the contours) or:

For 30 cm CI, the required spot height accuracy, $RMSE_V = 1/6 * 30$ cm $= 5$ cm

Data with $RMSE_V = 10$ cm would support Class 2 accuracy for spot elevations at this contour interval.

B.6 Relating ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) Values to Legacy NMAS 1947 Accuracy Values

In this section, examples are provided for users who wish to relate these Standards to the legacy National Map Accuracy Standards (NMAS) of 1947.

The legacy 1947 Standards use two accuracy criteria based on map scale: "1/30 inch for map scales larger than 1:20,000" and "1/50 inch for maps with a scale of 1:20,000 or smaller." Here horizontal accuracy refers to the Circular Map Accuracy Standard (CMAS) or Circular Error at the 90% Confidence Level (CE90).

Regarding vertical accuracy, the legacy 1947 Standards state:

> *"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."*

Here vertical accuracy refers to the Vertical Map Accuracy Standard (VMAS) or Linear Error at the 90% Confidence Level (LE90).

The following examples illustrate the procedures users can follow to relate horizontal and vertical accuracy values between ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) and the legacy 1947 Standards.

Example 3: Relating the Horizontal Accuracy of a Map or Orthorectified Image calculated with the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) to the Legacy National Map Accuracy Standards of 1947

Given a map or orthoimagery with a horizontal accuracy class of $RMSE_H$= 15-cm according to these Standards, compute the equivalent accuracy and map scale according to the legacy 1947 Standards.

Solution:

The horizontal accuracy class of $RMSE_H$ = 15-cm is representative of data sets typically used to create large-scale maps, so for this example, we will apply the criterion for scales larger than 1:20000.

Use the factor "1/30 inch":

$$RMSE_H = \sqrt{RMSE_X{}^2 + RMSE_Y{}^2}$$

or

$RMSE_X$ *or* $RMSE_Y = RMSE_H / 1.414$

Therefore,

CMAS (CE90) = 2.1460 * $RMSE_X$ = 2.1460 * $RMSE_Y$ = 2.1460 * $RMSE_H$ /1.414 = 1.5175 * $RMSE_H$

CMAS (CE90) = 1.5175 * 15 cm = *22.76 cm*

Convert CE90 to feet:

22.76 cm = *0.76 ft*

Use the NMAS accuracy relation of CE90 = 1/30" on the map to compute the map scale:

CE90 = 1/30 * ground distance covered by an inch of the map, or

ground distance covered by an inch of the map = CE90 * 30

ground distance covered by an inch of the map = 0.76 ft * 30 = *22.76 ft*

The equivalent map scale according to NMAS is 1" = 22.76', or 1:273

Example 4: Relating the Vertical Accuracy of an Elevation Data Set calculated with the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, v2, 2024 to the Legacy National Map Accuracy Standards of 1947

Given an elevation data set with a vertical accuracy class of $RMSE_V$ = 10-cm according to these Standards, compute the equivalent contour interval according to the legacy 1947 Standards.

Solution:

As mentioned earlier, the legacy 1947 Standards state that:

"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."

Compute error at 90% confidence using $RMSE_V$:

VMAS (LE90) = 1.6449 * $RMSE_V$ = 1.6449 * 10 cm = *16.449 cm*

Compute the contour interval (CI) using the following criteria set by the NMAS:

VMAS (LE90) = ½ CI, or

CI = 2 * LE90 = 2 * 16.449 cm = *32.9 cm*

B.7 Relating ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) Values to the FGDC National Standard for Spatial Data Accuracy (NSSDA)

In this section, examples are provided for users who wish to relate these Standards to the FGDC National Standard for Spatial Data Accuracy (NSSDA).

Example 5: Relating the Horizontal Accuracy of a Map or Orthorectified Image calculated with ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) to the FGDC National Standard for Spatial Data Accuracy (NSSDA)

Given a map or orthoimagery with a horizontal accuracy class of $RMSE_H$ = 15-cm according to the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, v2, 2024, compute the equivalent accuracy and map scale according to the FGDC National Standard for Spatial Data Accuracy (NSSDA).

Solution:

According to NSSDA, the horizontal positional accuracy is estimated at 95% confidence level using the following formula:

$Accuracy_{H95\%} = 1.7308 * RMSE_H = 1.7308 * 15\ cm = \mathit{25.96\ cm}$

Example 6: Relating the Vertical Accuracy of an Elevation Data Set calculated with the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024) to the FGDC National Standard for Spatial Data Accuracy (NSSDA)

Given an elevation data set with a vertical accuracy of $RMSE_V$ = 10-cm according to the Edition 2, Version 2 (2024), compute the vertical accuracy according to the FGDC National Standard for Spatial Data Accuracy (NSSDA).

Solution:

According to NSSDA, the vertical accuracy of an elevation data set is estimated at 95% confidence level using the following formula:

$Accuracy_{V95\%} = 1.96 * RMSE_V$

$Accuracy_{V95\%} = 1.96 * 10\ cm = \mathit{19.60\ cm}$

B.8 Estimating Horizontal Accuracy of Lidar Data

As described in Section 7.6, the horizontal error component of lidar is largely a function of GNSS positional error, IMU angular error, and flying height. These are not the only contributing factors to horizontal error, but, taken together, they can provide a rough estimate of the total horizontal error, which can be helpful when planning data acquisition where horizontal accuracy is a concern.

If the radial horizontal positional error of the GNSS is assumed to be equal to 0.10 m (based on 0.07 m in either X or Y), and the IMU error is assumed to be 10.0 arc-seconds (0.0027 degrees) for the roll and pitch and 15.0 arc-seconds (0.00417 degree) for the yaw/heading, Table B.8 can be used to predict the horizontal accuracy of the lidar point ($RMSE_H$) captured within a 40-degree field of view at different flying heights above mean terrain (FH).

Table B.8 Estimated Horizontal Error ($RMSE_H$) as a Function of GNSS Error, IMU Error, and Flying Height

Flying Height (m)	GNSS Error (cm)	IMU Roll/Pitch Error (arc-sec)	IMU Heading Error (arc-sec)	$RMSE_H$ (cm)
500	*10*	*10*	*15*	*10.7*
1,000	*10*	*10*	*15*	*12.9*
1,500	*10*	*10*	*15*	*15.8*
2,000	*10*	*10*	*15*	*19.2*
2,500	*10*	*10*	*15*	*22.8*
3,000	*10*	*10*	*15*	*26.5*
3,500	*10*	*10*	*15*	*30.4*
4,000	*10*	*10*	*15*	*34.3*
4,500	*10*	*10*	*15*	*38.2*
5,000	*10*	*10*	*15*	*42.0*

Each lidar system has its own specifications for GNSS and IMU error; therefore, the values in Table B.8 should be modified according to the equation in Section 7.6.

B.9 Elevation Data Accuracy vs. Elevation Data Quality

In aerial imaging and photogrammetry, the horizontal and vertical accuracy of individual points are largely dependent on the scale and resolution of the source imagery. Larger-scale imagery flown at a lower altitude produces smaller GSD and higher measurement accuracy. Users have, quite naturally, come to equate higher-resolution imagery (smaller GSD) with higher accuracy and higher quality.

In airborne topographic lidar, this is not entirely the case. For many typical lidar collections, the maximum accuracy attainable is limited by the combined error budget for all components of the lidar system, including laser ranging error, GNSS positional error, IMU angular error, and encoder error. Increasing the resolution of the data by increasing point density does not change the system error. Beyond the lidar system, the data must also be properly controlled, calibrated, boresighted, and processed. Errors introduced during any of these steps will affect the accuracy of the data, regardless of the point density. That said, high density lidar data is usually of higher *quality* than low density data, and the increased quality can manifest as *apparently* higher accuracy.

To accurately represent a complex terrain surface, higher point density is required to capture surface details and linear features, such as curbs and micro drainage features. In vegetated areas, where many lidar pulses are fully reflected before reaching the ground, a higher density data set tends to be more accurate because more points will penetrate through. More ground points will result in more accurate interpolation between points and, thus, improved surface definition. The need for dense ground points is greatest in variable or complex surfaces, such as mountainous terrain, where generalized interpolation between points would not accurately model all changes in the surface.

However, while the use of denser data for complex surface representation improves the accuracy of the derived surface at locations between the lidar measurements, it does not necessarily make the individual lidar measurements any more accurate. For more details on the topic, consult Addendum I of these Standards.

Increased density may not significantly improve the accuracy of the terrain model in flat, open terrain where interpolation between points may still adequately represent the ground surface. However, higher density data may still improve the quality of the data by adding additional detail to the final surface model, improving detection of edges for breaklines and increasing the confidence of the relative accuracy in swath overlap areas by reducing interpolation within the data set. High density data collection will also produce higher resolution lidar intensity images, which is always useful when using intensity data to aid in interpretation, edge detection, and feature extraction.

APPENDIX C – ACCURACY TESTING AND REPORTING GUIDELINES (NORMATIVE)

C.1 Checkpoint Requirements

Checkpoints used to assess product accuracy should be derived from an independent set of points that was not already used in processing or calibrating the product under evaluation. Checkpoints should have higher accuracy than the product being evaluated; they can be either field surveyed or derived from another product of higher accuracy.

The total number of points and their spatial distribution are both important in accuracy assessment. Legacy standards and guidelines typically specified a minimum number of checkpoints, and, in some cases, the type of land cover where they were to be acquired, but they did not define or characterize the spatial distribution of the points. A quantitative methodology for characterization and specification of the spatial distribution of checkpoints which accounts for land cover type and project shape does not currently exist. ASPRS encourages research into this topic for future revisions of these Standards. In the interim, this Appendix provides general recommendations and guidelines for quantity and placement of checkpoints for accuracy assessment.

For surveying the checkpoints, users of these Standards are encouraged to follow the best practices and guidelines presented in Addendum II, as these are based on industry consensus. Additionally, when conducting accuracy tests, users are advised to follow the guidelines presented in Addendum I, Section B to evaluate error normality, and to evaluate whether the errors are normally distributed and are well modeled by a Gaussian distribution. Testing error normality can sometimes help detect problems in checkpoints used for the accuracy evaluation and therefore in the validity of the test.

C.2 Accuracy of Checkpoints

According to these Standards, checkpoints should be at least twice the accuracy of the final product specification. Checkpoints of suspect quality should not be used for product accuracy assessment. Individual checkpoints showing errors larger than $3 * RMSE_{H_1}$ or $3 * RMSE_{V_1}$ should be investigated. Addendum II of these Standards should be consulted when surveying checkpoints.

C.3 Number of Checkpoints

Table C.1 lists ASPRS recommendations for the number of checkpoints to be used for the horizontal accuracy testing of digital orthoimagery and planimetric data sets, and for the horizontal and vertical accuracy of elevation data. For vertical accuracy testing, users of the Standards should follow these recommendations:

Testing Non-Vegetated Vertical Accuracy (NVA): The number of checkpoints should be based on Table C.1.

Testing Vegetated Vertical Accuracy (VVA): If the project requires the VVA to be tested, an additional 30 checkpoints should be acquired to evaluate vertical accuracy of the terrain elevation data within vegetation. These 30 checkpoints should be well-distributed across the vegetated area of the project. The data user and data producer may agree to collect a larger number of checkpoints. To avoid situations where the errors in checkpoints in vegetated terrain may not follow a random distribution, users are advised to examine the individual elevation differences (i.e., errors) for each checkpoint in addition to the RMSE and other statistical terms computed in the test. Sections B and C of Addendum I should be consulted when evaluating the results of the VVA assessment. These two sections contain information regarding error distribution and the presence of biases in the data.

The project area should be divided into non-vegetated areas and vegetated areas. Then, the appropriate number of checkpoints should be acquired in each area to test the horizontal accuracy of the digital orthophotos and planimetric data and the vertical accuracy of the elevation data. For the non-vegetated areas, appropriate checkpoint quantity should be extracted from Table C.1. For vegetated areas, an additional 30 checkpoints should be acquired to evaluate vertical accuracy of elevation data.

For projects where the 30-checkpoint minimum is not feasible (e.g., the project is testing too small of an area, or is working under budget constraints), an accuracy verification using a smaller number of checkpoints should be reported according to the statements provided in Section 7.16 of these Standards.

Table C.1 Recommended Number of Checkpoints for Horizontal Accuracy and NVA Testing Based on Project Area

Project Area (Square Kilometers)	Total Number of Checkpoints for NVA
≤1000[1]	*30*
1001–2000	*40*
2001–3000	*50*
3001–4000	*60*
4001–5000	*70*
5001–6000	*80*
6001–7000	*90*
7001–8000	*100*
8001–9000	*110*
9001–10000	*120*
>10000	*120*

The recommended number and distribution of NVA and VVA checkpoints may vary depending on the importance of different land cover categories and project requirements. The checkpoint quantities put forward in Table C.1 are only recommendations based on best practices. Data producers and data users

[1] For very small projects where the use of 30 checkpoints is not feasible, report the accuracy as suggested in Section 7.16.

may agree to alter such requirements based on expected accuracy, project area and scope, terrain difficulties, area accessibility, and budget.

C.4 Distribution of Vertical Checkpoints across Land Cover Types

The recommended number of checkpoints should be distributed evenly around the vegetated and non-vegetated areas of the project. There may be exceptions depending on the nature of the terrain and land cover; however, efforts should be made to assure that the best possible checkpoint distribution is achieved.

ASPRS recognizes that some project areas are primarily non-vegetated, while other areas are primarily vegetated. For these reasons, the distribution of checkpoints can vary based on the general proportion of vegetated to non-vegetated areas in the project area. Checkpoints should generally be distributed proportionally among the various vegetated land cover types in the project. In areas where difficult terrain and transportation limitations may render some land cover types inaccessible, the desired spatial distribution of checkpoints across land cover types may not be possible. In these situations, data producers should consult with their users and use their best professional judgment in selecting checkpoint locations.

The recommendations in Sections C.1 through C.3 intentionally offer a fair amount of discretion in the location and distribution of checkpoints on the parts of the data user and producer. The general location and distribution of checkpoints should be discussed and agreed upon by the data user and producer as part of the project plan.

C.5 Vertical Checkpoints

Vertical checkpoints do not need to be well-defined point features; however, they should be placed on smooth, level or gently sloping terrain away from natural breaks and above-ground features such as curbs, bushes, trees, or in locations like parking lots where cars may be parked during aerial data acquisition. Surveying equipment and methodology should be selected based upon the accuracy needs of the final product; general best practices and guidelines for surveying are addressed in detail in Addendum II.

Vertical checkpoints should be at least 2 times more accurate than the required accuracy of the elevation data set being tested.

C.6 Horizontal Checkpoints for Elevation Data

Elevation data sets do not always contain the type of well-defined points that are required for horizontal testing according to these Standards. Specific methods for testing and verifying horizontal accuracies of elevation data sets depend on the technology used and the project design. The specific testing methodologies should be identified in the metadata.

The horizontal accuracy of elevation data generated from photogrammetric processes is the same as the horizontal accuracy achieved for orthophotos or for planimetric maps generated from the same aerial triangulation.

For horizontal accuracy testing of lidar data sets, it is recommended that at least half of the NVA vertical checkpoints should be located at point features visible on the lidar intensity image, as this allows them to also serve as horizontal checkpoints. The intersection of paint stripes on concrete or asphalt surfaces are normally visible on lidar intensity images, as are 90-degree corners of different reflectivity, e.g., a sidewalk corner adjoining a grass surface. The data producer is responsible for establishing methodologies appropriate to the technologies used to verify that horizontal accuracies meet the stated requirements.

Testing the horizontal accuracy of lidar data is often difficult, and thus is not always performed. In most cases, users follow the lidar system manufacturer's estimation of horizontal accuracy, as there are no good alternatives. Section 7.6 provides a formula for estimation of horizontal accuracy as a function of flying height for given sensor parameters, which can be useful for planning lidar data acquisition missions when horizontal accuracy is a concern.

C.7 Testing and Reporting of Product Accuracy

New in Edition 2 of these Standards is the inclusion of checkpoint survey error in the final computation of the product accuracy. Mapping technologies today can produce data with accuracy that approaches the accuracy of GPS surveys; therefore, two components of error must be accounted for in product testing. The first component of error is caused by the inaccuracy of the internal geometric determination during the aerial triangulation of imagery, or the boresight calibration in lidar processing. The second component of error is introduced by the auxiliary systems used, such as GPS or IMU, or by the instruments used for the ground control and checkpoint surveying. The latter error results in erroneous datum estimation. To accurately compute the product's $RMSE_H$, $RMSE_V$, or $RMSE_{3D}$, the error from the mathematical modeling and calibration, as well as the error in the datum estimation due to inaccurate ground control or checkpoints, should be considered. The following formula represents the updated and accepted method for computing product accuracy:

$$Horizontal\ Product\ Accuracy\ (RMSE_H) = \sqrt{{RMSE_{H_1}}^2 + {RMSE_{H_2}}^2}$$

$$Vertical\ Product\ Accuracy\ (RMSE_V) = \sqrt{{RMSE_{V_1}}^2 + {RMSE_{V_2}}^2}$$

$$RMSE_{3D} = \sqrt{{RMSE_H}^2 + {RMSE_V}^2}$$

Where:

$RMSE_H$, $RMSE_V$, and $RMSE_{3D}$ are the product's horizontal accuracy, vertical accuracy, and three-dimensional accuracy respectively.

$RMSE_{H_1}$ and $RMSE_{V_1}$ are the components of error derived from product fit to the checkpoints.

$RMSE_{H_2}$ and $RMSE_{V_2}$ are the components of error associated with the checkpoint surveys.

As an example, compute the vertical accuracy of mobile lidar data set using independent checkpoints according to the above formula given the following:

- The survey report states that the RTK techniques produced checkpoints with $RMSE_{V_2}$ = 3-cm.

- When the checkpoints were used to verify the vertical accuracy of the lidar data, the fit of the lidar data to the checkpoints was found to be $RMSE_{V_1}$ = 1-cm; see Section D.1.1 on how to calculate $RMSE_{V_1}$.

Using the formula above:

$$Vertical\ Product\ Accuracy = \sqrt{1^2 + 3^2} = 3.16\ cm$$

The correct vertical accuracy of the lidar dataset with respect to the vertical datum is 3.16 cm, rather than the commonly reported value of 1 cm. Additional examples of accuracy computation can be found in Appendix D.

C.7.1 Testing and Reporting Horizontal Accuracy of Digital Orthophotos and Planimetric Maps

For testing and reporting the horizontal accuracy of digital orthophoto and planimetric maps, ASPRS endorses the use of $RMSE_H$ alone, provided that the horizontal errors are normally distributed, the sample size is sufficiently large, and the mean error is sufficiently small. The horizontal accuracy of these products is primarily determined by the accuracy of the aerial triangulation solution. In testing horizontal accuracy, poor point selection or poor measurement techniques can add additional error to the accuracy assessment results. When measuring checkpoints, users should zoom to the highest level possible to minimize pointing errors; ideally, a zoom level that results in sub-pixel pointing accuracy is desirable. If this is not possible or was not practiced, pointing error should be factored into the product accuracy assessment.

Example: Assume that a technician was tasked to assess the horizontal accuracy of an orthophoto of 10 cm GSD. The data was produced to meet the ASPRS Horizontal Accuracy Class of 20-cm. Additionally, assume that, for whatever reason, the technician performed the measurements at a zoom level that introduces 2-pixel pointing error. The “tested to meet” horizontal accuracy as reported by the technician should be as follows:

$$RMSE_H = \sqrt{(20.0)^2 + (2 * 10.0)^2} = 28.28\ cm$$

In this case, the product accuracy is better than the “tested to meet” accuracy, because measurement error was introduced during the testing process. If the “tested to meet” horizontal accuracy does not meet or exceed the “produced to meet” horizontal accuracy, consideration should be given for this additional source of error before determining whether or not the project has been completed to specification.

C.7.2 Testing and Reporting of Vertical Accuracy of Elevation Data

For testing and reporting the vertical accuracy of digital elevation data, ASPRS endorses the use of $RMSE_V$ alone, provided that the vertical errors are normally distributed, the sample size is sufficiently large, and the mean error is sufficiently small.

VVA should also be computed using $RMSE_V$, but its evaluation should also consider the reported individual elevation differences (i.e., errors) for each checkpoint. Care should be taken when evaluating skew; skewed results may occur in vegetated areas due to the low density of the lidar point cloud and the degraded quality of GPS surveys under trees. By testing and reporting the VVA separate from the

NVA, ASPRS draws a clear distinction between non-vegetated terrain and vegetated terrain where data may be less accurate. These Standards rely primarily on lidar performance in open and unobscured terrain when evaluating data accuracy and quality.

C.8 Low Confidence Areas

For stereo-compiled elevation data sets, photogrammetrists should capture two-dimensional closed polygons for low confidence areas where the bare-earth DTM may not meet the overall data accuracy requirements. Because photogrammetrists cannot see the ground in stereo beneath dense vegetation, in deep shadows, or where the imagery is otherwise obscured, reliable data cannot be collected in those areas. Traditionally, contours within these obscured areas would be published as dashed contour lines. A compiler should make the determination as to whether the data being digitized is within vegetated or non-vegetated areas. Areas not delineated by an obscure area polygon are assumed to meet accuracy standards. The extent of photogrammetrically derived obscure area polygons and any assumptions regarding how NVA and VVA accuracies apply to the photogrammetric data set must be clearly documented in the metadata.

Low confidence areas also occur with lidar and IFSAR where heavy vegetation causes poor penetration of the lidar pulse or radar signal. Low confidence areas can be identified with raster analysis based on the following four criteria and converted into 2D polygons for delivery:

- Nominal ground point density (NGPD)
- Search radius to determine average ground point density
- Cell size for the raster analysis
- Minimum size of generalized low confidence areas (minimum mapping unit)

This section describes possible methods for the collection or delineation of low confidence areas in elevation data sets. This list is not meant to be exhaustive; other methodologies currently exist, and additional techniques will certainly emerge in the future. The data producer may use any method they deem suitable, provided the technique is well documented in the metadata.

Table C.2 gives recommendations for low confidence criteria as they relate to Vertical Accuracy Class based on the following assumptions:

- *Nominal Ground Point Density (NGPD):* Areas with ground point densities less than or equal to ¼ of the recommended nominal pulse density (NPD) are candidates for low confidence areas. For example, a project specification calls for NPD of 1 pt/m^2, but in some vegetated areas, the NGPD is 0.25 pt/m^2. Such areas are good candidates for low confidence polygons.
- *Search Radius:* A search area with radius equal to 3 * NPS for the project (not the low confidence NGPD). This radius is small enough to allow good definition of low-density areas while not being so small as to cause the project to look worse than it actually is.
- *Raster Analysis Cell Size:* To facilitate raster analysis, use a cell size equal to the search radius.
- *Minimum Size for Low Confidence Polygons:* The areas computed with low densities should be aggregated together. Unless specifically requested by the data user, structures/buildings and

water should be removed from the aggregated low-density polygons, as these features do not represent true low confidence areas. Aggregated polygons greater than or equal to the stated minimum size as provided in Table C.2 should be kept and defined as low confidence polygons. In some cases, too many small areas will “checkerboard” the low confidence areas; in other cases, too many large areas will not adequately delineate low confidence areas. The minimum size of low confidence polygons should be determined by the topography, land cover, and the intended purpose of the maps.

Table C.2 Low Confidence Area Criteria
Min NPD: Minimum Nominal Point Density, Max NPS: Maximum Nominal Point Spacing
Min NGPD: Minimum Nominal Ground Point Density, Max NGPS: Maximum Nominal Ground Point Spacing

Vertical Accuracy Class	Project Min NPD (pts/m^2) [Max NPS (m)]	Low Confidence Min NGPD (pts/m^2) [Max NGPS (m)]	Cell Size for Computing NGPD (m)	Low Confidence Polygon Minimum Size (acres) [m^2]
1-cm	*≥ 20 [≤ 0.22]*	*≥ 5 [≤ 0.45]*	*0.67*	*0.5 [2,000]*
2.5-cm	*≥ 16 [≤ 0.25]*	*≥ 4 [≤ 0.50]*	*0.75*	*1 [4,000]*
5-cm	*≥ 8 [≤ 0.35]*	*≥ 2 [≤ 0.71]*	*1.06*	*2 [8,000]*
10-cm	*≥ 2 [≤ 0.71]*	*≥ 0.5 [≤ 1.41]*	*2.12*	*5 [20,000]*
15-cm	*≥ 1.0 [≤ 1.0]*	*≥ 0.25 [≤ 2.0]*	*3.0*	*5 [20,000]*
20-cm	*≥ 0.5 [≤ 1.4]*	*≥ 0.125 [≤ 2.8]*	*4.24*	*5 [20,000]*
33.3-cm	*≥ 0.25 [≤ 2.0]*	*≥ 0.0625 [≤ 4.0]*	*6.0*	*10 [40,000]*
66.7-cm	*≥ 0.1 [≤ 3.2]*	*≥ 0.025 [≤ 6.3]*	*9.5*	*15 [60,000]*
100-cm	*≥ 0.05 [≤ 4.5]*	*≥ 0.0125 [≤ 8.9]*	*13.4*	*20 [80,000]*
333.3-cm	*≥ 0.01 [≤ 10.0]*	*≥ 0.0025 [≤ 20.0]*	*30.0*	*25 [100,000]*

Acres should be used as the unit of measurement for the low confidence area polygons, as many agencies (USGS, NOAA, USACE, etc.) use acres as the mapping unit for required polygon collection.

Approximate square meter equivalents are provided for those whose work is exclusively in the metric system. Smoothing algorithms can be applied to the low confidence polygons, if desired.

There are two distinct types of low confidence areas:

- The first type is identified by the data producer in advance. Areas where acceptable representation of bare earth is expected to be unlikely or impossible should be accounted for ahead of time. No ground control or checkpoints should be located in these areas, and contours, if produced, should be dashed. These areas are exempt from accuracy assessment. Mangroves, swamps, and inundated wetland marshes are prime candidates for this type of delineation.
- The second type occurs in valid VVA areas, such as forests where checkpoints should be surveyed and accuracy assessments ought to be performed, even if they would traditionally be depicted with dashed contours. Such low confidence areas are delineated after classification and are usually identifiable by notably low-density bare-earth points.

If reliable elevation data in low confidence areas is critical to a project, it is common practice to supplement the remote sensing data with field surveys.

C.9 Erroneous Checkpoints

Occasionally, a checkpoint may be—at no fault of the lidar survey—erroneous or inappropriate for use. Such points may be removed from the accuracy assessment calculation if they meet one or more of the following criteria:

- If it is demonstrated via pictures and descriptions that the checkpoint was improperly located, such as a vertical checkpoint that had been placed on steep terrain or within a few meters of a significant breakline that redefines the slope of the area interpolated surrounding the checkpoint.
- If it is demonstrated and documented that the topography has changed significantly between the time the elevation data was acquired and the time the checkpoint was surveyed.
- If (a) the point is included in the survey and accuracy reports, but not the assessment calculation, with pictures and descriptions; (b) reasonable efforts to correct the discrepancy are documented (e.g., rechecked airborne GNSS and IMU data, rechecked point classifications in the area, rechecked the ground checkpoints); and (c) a defensible explanation is provided in the accuracy report for discarding the point.

An explanation that the error exceeds 3 times the standard deviation ($> 3\sigma$) is NOT an acceptable explanation without the proper investigations or justifications as described above.

C.10 Data Internal Precision Assessment

To the greatest extent possible, the location of the test points for determining the data internal precision should meet the following criteria:

- include all overlap areas (sidelap, endlap, and cross flights)
- be evenly distributed throughout the full width and length of each overlap area
- be in non-vegetated areas (clear and open terrain, urban areas, etc.)

- be at least three (3) meters away from any vertical artifact or abrupt change in elevation
- be on uniform slopes
- not include points that are determined to be invalid surface returns, including points with poor geometry

While RMS_{DZ} may be calculated using a set of specific test location points, the maximum difference requirement is not limited to these locations: it applies to all locations within the entire data set that meet the above criteria.

C.11 Interpolation of Elevation Represented Surface for Checkpoint Comparisons

The surface representation of an elevation data set is normally a TIN (Figure C.1) or a raster DEM (Figure C.2).

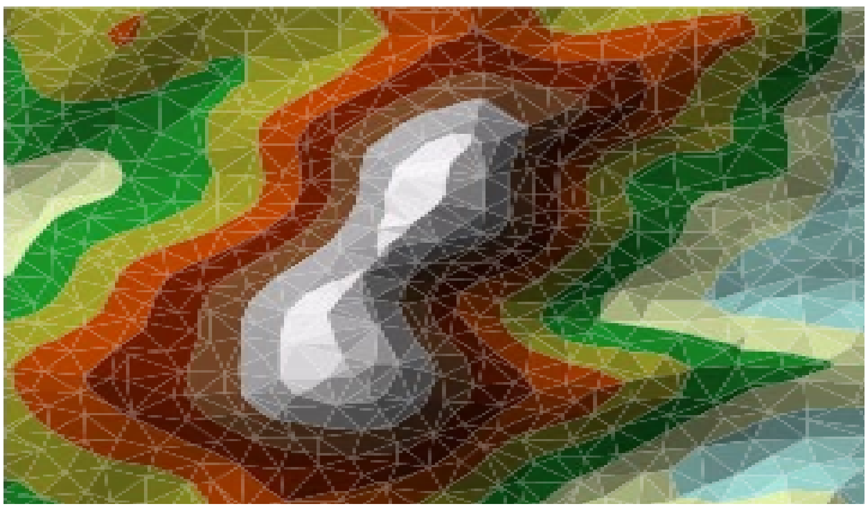

Figure C.1 Topographic Surface Represented as a TIN

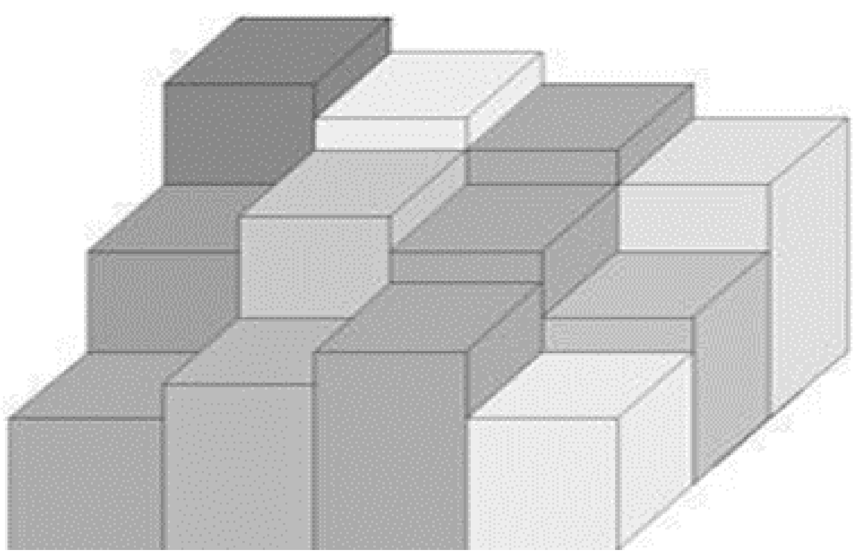

Figure C.2 Topographic Surface Represented as a DEM

Vertical accuracy testing is accomplished by comparing the elevation of the represented surface of the elevation data set to elevations of checkpoints at the horizontal (X, Y) coordinates of the checkpoints. The data set surface is most often represented by a TIN or raster DEM.

Vertical accuracy of point-based elevation data sets should be tested by first creating a TIN from the point-based elevation data set, and then comparing the TIN elevations to the checkpoint elevations. TINs should be used to test the vertical accuracy of point-based elevation data sets because it is unlikely a checkpoint will be located at the location of a discrete elevation point. The TIN methodology is commonly used for interpolating elevations from irregularly spaced point data. Other potentially more accurate methods of interpolation exist, and these may be addressed by future versions of these Standards as they become more commonly used and accepted.

Vertical accuracy of raster DEMs should be evaluated by comparing the elevation of the DEM, which is already a continuous surface, to the checkpoint elevations. For most DEM data sets, it is recommended that the elevation of the DEM be determined by extracting the elevation of the pixel that contains the XY coordinates of the checkpoint. However, in some instances, such as when the DEM being tested is at a lower resolution than is typical of global data sets or when the truth data has an area footprint associated with it rather than a single XY coordinate, it may be better to use interpolation methods to determine the elevation of the DEM data set.

Data producers should seek approval from data users if methods other than extraction are to be used to determine elevation values of the DEM data set. Vertical accuracy testing methods listed in the metadata and the reports should state if elevation values were extracted from the tested data set at the XY location of the checkpoints, or if further interpolation was used after the creation of the tested surface (TIN or raster) to determine the elevation of the tested data set. If further interpolation was used, the interpolation method and full process used should be detailed accordingly.

APPENDIX D – ACCURACY STATISTICS AND EXAMPLES (NORMATIVE)

D.1 Reporting Accuracy Statistics

The National Standard for Spatial Data Accuracy (NSSDA) documents the equations for the computation of $RMSE_X$, $RMSE_Y$, $RMSE_R$, and $RMSE_Z$, as well as horizontal (radial) and vertical accuracies at the 95% confidence levels—$Accuracy_R$ and $Accuracy_Z$, respectively. These statistics assume that errors approximate a normal error distribution and that the mean error is small relative to the target accuracy. The ASPRS Positional Accuracy Standards for Digital Geospatial Data reporting methodology is based on RMSE alone as the accuracy measure, and thus differs from the NSSDA reporting methodology. Additionally, these ASPRS Standards include surveying error inherited from ground control and checkpoints in the computed final product accuracy, as discussed in Appendix C. In addition to RMSE, these Standards require computing additional statistical terms that may be used to investigate the pattern of errors in data. These statistical terms are mean, median, and standard deviation. It is also recommended to plot the error vectors for each checkpoint, as this can help diagnose systematic error or patterns of errors.

D.1.1 Accuracy Computations

For the purposes of demonstration, suppose you must use 5 checkpoints to verify the final horizontal and vertical accuracy for a data set (this is fewer than the 30 checkpoints required by these Standards—the example uses fewer for the sake of brevity) according to these Standards.

Table D.1 provides the map-derived coordinates and the surveyed coordinates for the 5 points. The table also shows the computed accuracy and other relevant statistics. In this abbreviated example, the data are intended to meet a target Horizontal Accuracy Class of $RMSE_H$ = 15-cm and a target Vertical Accuracy Class of $RMSE_V$ = 10-cm.

Table D.1 Accuracy Statistics for Example Data

Point ID	Map-derived Values			Surveyed Checkpoints Values			Residuals (Errors)		
	Easting (E)	Northing (N)	Elevation (Z)	Easting (E)	Northing (N)	Elevation (Z)	ΔE (Easting)	ΔN (Northing)	ΔZ (Elevation)
	meter	meter	meter	meter	meter	meter	meter	meter	meter
GCP1	359584.394	5142449.934	477.127	359584.534	5142450.004	477.198	-0.140	-0.070	-0.071
GCP2	359872.190	5147939.180	412.406	359872.290	5147939.280	412.396	-0.100	-0.100	0.010
GCP3	359893.089	5136979.824	487.292	359893.072	5136979.894	487.190	0.017	-0.070	0.102
GCP4	359927.194	5151084.129	393.591	359927.264	5151083.979	393.691	-0.070	0.150	-0.100
GCP5	372737.074	5151675.999	451.305	372736.944	5151675.879	451.218	0.130	0.120	0.087
						Number of check points	5	5	5
						Mean Error (m)	-0.033	0.006	0.006
						Standard Deviation (m)	0.108	0.119	0.091
						RMSE (m)	0.102	0.106	0.081
						Fit to Checkpoints $RMSE_{H1}$ (m)	0.147	$RMSE_H = \sqrt{RMSE_E^2 + RMSE_N^2}$	
						Fit to Checkpoints $RMSE_{V1}$ (m)	0.081		

Computation of Horizontal, Vertical, and Three-Dimensional Accuracy:

1. Compute the Root-Mean-Square Error Values:

$$RMSE_x = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(x_{i(map)} - x_{i(surveyed)})^2}$$

where:

$x_{i(map)}$ is the coordinate in the specified direction of the *i*th checkpoint in the data set,

$x_{i(surveyed)}$ is the coordinate in the specified direction of the *i*th checkpoint in the independent source of higher accuracy,

n is the number of checkpoints tested,

and *i* is an integer ranging from 1 to *n.*

$$RMSE_x = \sqrt{\frac{(-0.140)^2+(-0.100)^2+(0.017)^2+(-0.070)^2+(0.130)^2}{5}} = 0.102\ m$$

$$RMSE_y = \sqrt{\frac{(-0.070)^2+(-0.100)^2+(-0.070)^2+(0.150)^2+(0.120)^2}{5}} = 0.107\ m$$

$$RMSE_{H_1} = \sqrt{{RMSE_x}^2 + {RMSE_y}^2}$$

$$RMSE_{H_1} = \sqrt{(0.102)^2 + (0.107)^2} = 0.147\ m$$

$$RMSE_{V_1} = \sqrt{\frac{(-0.071)^2+(0.010)^2+(0.102)^2+(-0.100)^2+(0.087)^2}{5}} = 0.081\ m$$

2. Compute the Final Accuracy Values:

To complete the accuracy computations, let us assume that the checkpoint report submitted by the surveyor states that the field survey was conducted using an RTK-GPS-based technique to an accuracy of:

$$Horizontal\ Accuracy\ RMSE_{H_2} = 1.9\ cm\ or\ 0.019\ m$$

$$Vertical\ Accuracy\ RMSE_{V_2} = 2.23\ cm\ or\ 0.022\ m$$

The final horizontal and vertical accuracy should be computed as follows:

$$RMSE_H = \sqrt{{RMSE_{H_1}}^2 + {RMSE_{H_2}}^2} = \sqrt{(0.147)^2 + (0.019)^2} = 0.148\ m\ (< 15\ cm)$$

$$RMSE_V = \sqrt{{RMSE_{V_1}}^2 + {RMSE_{V_2}}^2} = \sqrt{(0.081)^2 + (0.022)^2} = 0.083\ m\ (< 10\ cm)$$

Similarly, the three-dimensional positional accuracy can be computed using the following formula:

$$RMSE_{3D} = \sqrt{{RMSE_H}^2 + {RMSE_V}^2}$$

Therefore, according to the project specifications, the product should meet a three-dimensional positional accuracy of:

$$RMSE_{3D} = \sqrt{RMSE_H{}^2 + RMSE_V{}^2} = \sqrt{(0.15)^2 + (0.10)^2} = \mathit{0.18\ m\ or\ 18.0\ cm}$$

While the actual three-dimensional positional accuracy is found to be:

$$RMSE_{3D} = \sqrt{0.148^2 + 0.083^2} = \mathit{0.170\ m\ (< 18\ cm)}$$

Based on the computed horizontal and vertical accuracy numbers above, the product is meeting the specified horizontal and vertical accuracies of 15-cm and 10-cm, respectively.

Computation of Mean Errors in X, Y, and Z:

$$\overline{x} = \frac{1}{(n)} \sum_{i=1}^{n} x_i$$

where:

$\overline{x}$ is the mean error in the specified direction,

x_i is the i^{th} error in the specified direction,

n is the number of checkpoints tested,

i is an integer ranging from 1 to *n.*

$$\textit{Mean error in Easting: } \overline{x} = \frac{-0.140 - 0.100 + 0.017 - 0.070 + 0.130}{5} = \mathit{-0.033\ m}$$

$$\textit{Mean error in Northing: } \overline{y} = \frac{-0.070 - 0.100 - 0.070 + 0.150 + 0.120}{5} = \mathit{0.006\ m}$$

$$\textit{Mean error in Elevation: } \overline{z} = \frac{-0.070 + 0.010 + 0.102 - 0.100 + 0.087}{5} = \mathit{0.006\ m}$$

Computation of Sample Standard Deviation:

$$s_x = \sqrt{\frac{1}{(n-1)} \sum_{i=1}^{n} (x_i - \overline{x})^2}$$

where:

x_i is the i^{th} error in the specified direction,

$\overline{x}$ is the mean error in the specified direction,

n is the number of checkpoints tested,

i is an integer ranging from 1 to *n.*

Sample Standard Deviation in Easting:

$$s_x = \sqrt{\frac{\big(-0.140-(-0.033)\big)^2 + \big(-0.100-(-0.033)\big)^2 + \big(0.017-(-0.033)\big)^2 + \big(-0.070-(-0.033)\big)^2 + \big(0.130-(-0.033)\big)^2}{(5-1)}} = \mathit{0.108\ m}$$

Sample Standard Deviation in Northing:

$$s_y = \sqrt{\frac{(-0.070-0.006)^2 + (-0.100-0.006)^2 + (-0.070-0.006)^2 + (0.150-0.006)^2 + (0.120-0.006)^2}{(5-1)}} = \mathit{0.119\ m}$$

Sample Standard Deviation in Elevation:

$$s_z = \sqrt{\frac{(-0.071-0.006)^2+(0.010-0.006)^2+(0.102-0.006)^2+(-0.100-0.006)^2+(0.087-0.006)^2}{(5-1)}} = 0.091\ m$$

Computation of Population Standard Deviation:

$$s_x = \sqrt{\frac{1}{(n)}\sum_{i=1}^{n}(x_i - \overline{x})^2}$$

where:

x_i is the *i^th^* error in the specified direction,

$\overline{x}$ is the mean error in the specified direction,

n is the number of checkpoints tested,

i is an integer ranging from 1 to *n.*

Population Standard Deviation in Easting:

$$s_x = \sqrt{\frac{(-0.140-(-0.033))^2+(-0.100-(-0.033))^2+(0.017-(-0.033))^2+(-0.070-(-0.033))^2+(0.130-(-0.033))^2}{(5)}} = 0.096\ m$$

Population Standard Deviation in Northing:

$$s_y = \sqrt{\frac{(-0.070-0.006)^2+(-0.100-0.006)^2+(-0.070-0.006)^2+(0.150-0.006)^2+(0.120-0.006)^2}{(5)}} = 0.106\ m$$

Population Standard Deviation in Elevation:

$$s_z = \sqrt{\frac{(-0.071-0.006)^2+(0.010-0.006)^2+(0.102-0.006)^2+(-0.100-0.006)^2+(0.087-0.006)^2}{(5)}} = 0.081\ m$$

ADDENDUM I: GENERAL BEST PRACTICES AND GUIDELINES

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This Addendum contains best practices and guidelines for all users of the ASPRS Positional Accuracy Standards for Digital Geospatial Data.

SECTION A: REPORTING NOTES FOR DELIVERED PRODUCTS

Contributor: Michael Zoltek, GPI Geospatial, Inc.

The ASPRS Positional Accuracy Standards for Digital Geospatial Data encourage truth in reporting when delivering geospatial products or services. This section provides examples of reporting notes to accompany delivered products. Subsections provide specific reporting guidelines for various categories of deliverables.

All accuracies should be reported as "tested to meet" or "produced to meet" in accordance with ASPRS Positional Accuracy Standards for Digital Geospatial Data, Section 7.16. To provide clients with the required metadata to support the proper use of geospatial deliverables, it is recommended that the following notes be included in reports for the various types of deliverables described herein:

A.1 Notes Related to Geospatial Deliverables in General

1. The elevation data provided was ***tested to meet a vertical accuracy of xxx (units) RMSE, using xxx checkpoints*** in clear, unobscured areas, to support the generation of a ***x-(units) contour interval.***
2. The delivered elevation data is the source for any delivered derivative products (e.g., contours). The project's delivered elevation data should be utilized as the sole source for creating any additional derivative products or subsequent computations.
3. This map was produced by photogrammetric methods using: (select all that apply)
    a. Aerial lidar
    b. Aerial photogrammetry
4. The following sensors were utilized to collect the data for this project:
    a. Aerial imagery sensor
        i. Sensor make
        ii. Sensor model
        iii. Calibration date
    b. Aerial lidar sensor
        i. Sensor make
        ii. Sensor model
        iii. Calibration date
5. The following software products were utilized during the creation of the deliverables:
    a. Trajectory processing
    b. Lidar data processing
        i. Calibration
        ii. Classification
        iii. Data extraction
        iv. Data validation
    c. Name and version of imagery processing software used:

            i. Aerial triangulation
            ii. Orthomosaic production
            iii. Stereo compilation
            iv. Data validation
    d. Name and version of software used for compilation:
            i. Stereo viewing/extraction
            ii. Lidar point cloud extraction
            iii. Data validation
6. Ground control and/or checkpoints were provided by:
    a. Firm name, address, phone number, and license number
    b. Signing surveyor name and license number
7. Ground control and/or checkpoint coordinate values are as follows:
    a. Provide coordinates in local State Plane or user-requested coordinate system.
    b. ALWAYS provide coordinates in lat/long/ellipsoid height to allow for validation of any coordinate transformations or reprojections.
8. GPS positional data was observed on/between the dates of ***mo/day/year*** and ***mo/day/year*** utilizing a ***make/model*** receiver. The grid coordinates of the Fixed Station(s) shown were derived using a ***describe network*** (e.g., Local Static Control, VRS network of CORS stations) referenced to ***datum (year), epoch (year), geoid (year)***.
9. The positional accuracy of the GPS vectors is: ***Horizontal H.HH (units), Vertical V.VV (units), Combined Grid Factor: 0.xxxxxxxx centered on Fixed Station xxxx as shown herein***.
10. Accuracies of horizontal control points are reported as being ***xxx (units) RMSE with a standard deviation of xxx (units)***. Individual point statistics can be found in ***Appendix X.*** *(Note: A Coordinate Quality report can be utilized to provide individual point statistics.)*
11. Accuracies of vertical control points are reported as being ***xxx (units) RMSE with a standard deviation of xxx (units)***. Individual point statistics can be found in ***Appendix X.***
12. Delivered products are referenced to the following spatial reference system:
    a. Horizontal datum with epoch
    b. Vertical datum with epoch and reference geoid
    c. Projection (UTM, State Plane, etc.)

A.2 Notes Related to Aerial Imagery Deliverables

1. Date(s) of Aerial Imagery Capture, ***mo/day/year.***

2. The imagery was ***collected at xxx (units) nominal GSD*** to support the production of orthorectified digital maps with ***xxx (units) GSD***.

3. The accuracy of aerial triangulation, which was performed ***using xxx ground control points and XYZ software/version***, was found to be ***$RMSE_H$ = xxx, $RMSE_V$ = yyy***. If aerial triangulation was not performed and the direct-georeferencing approach used instead, the accuracy of the IMU orientation angle, roll, pitch, and heading alongside the final product accuracy (as verified by checkpoints or ground control points, if available) should be reported.

4. Describe the source(s) of the elevation surface utilized to produce the orthophotography, as well as any modifications made to the orthophotography by the consultant. If multiple DEM sources were utilized, provide a description and graphical representation (or CADD or shapefiles) delineating areas covered by each DEM source.
5. This imagery mapping product was ***tested to meet a horizontal accuracy of xxx (units) RMSE$_H$ using xxx checkpoints.***
6. If a client specifies a legacy standard, add a comparison to the legacy equivalent, e.g., ***"which is equivalent to the ASPRS Accuracy Standards for Large-Scale Maps (1990) ASPRS Class 1 at a map scale of 1:2400."***
7. Compiled vector features ***have been tested to meet a horizontal accuracy of x.xx (units) RMSE, using xxx checkpoints*** in clear unobscured areas. Planimetric features in areas delineated as "visually obscured" may not adhere to this accuracy.
8. Compiled vector features ***have been tested to meet a vertical accuracy of x.xx (units) RMSE, using xxx checkpoints*** in clear unobscured areas. Planimetric features that lie in areas delineated as "visually obscured" may not adhere to this accuracy.
9. Report sequence of orientation angles: ***The exterior orientation angles rotation sequence is:***
   a. ω, ϕ, κ
   b. Other sequences
10. Report camera integration on aircraft: ***The camera was oriented with the image positive y-axis in the direction of flight***.

A.3 Notes Related to Aerial Lidar Deliverables

1. Date of Lidar Capture, ***mo/day/year.***
2. Lidar data was collected nominally at ***xxx points per square meter (or xxx points per square foot)*** resulting in an equivalent ***xxx cm (or xxx foot) nominal point spacing***.
3. This lidar mapping product ***was tested to meet a vertical accuracy of xxx (units) RMSE$_V$ using xxx checkpoints in non-vegetated terrain***.

SECTION B: ERROR NORMALITY TESTS

Contributor: Dr. Christopher E. Parrish, Oregon State University

B.1 Creating the Normality Test

Following an accuracy test, it is considered good practice to assess and report whether the errors[1] are normally distributed (i.e., whether they are well modeled by a Gaussian distribution). This assessment can provide context to the accuracy test results, and, in some cases, may help detect sources of error or

[1] In keeping with the terminology convention used throughout these Standards, in this Addendum, we use the term "errors" where, strictly-speaking, we mean "residuals."

other issues in the project framework. For example, if the error distribution is non-normal, this could indicate the presence of blunders or large systematic errors, which should be investigated further.

The first step in testing the normality of the error distribution is a visual test, which is performed by plotting and inspecting a histogram of errors. Histogram plotting functions are available in any number of spreadsheet software packages and programming languages. An example of an error histogram is shown in Figure I.B.1. This example is from testing the accuracy of an airborne lidar point cloud covering a portion of the Oregon State University (OSU) campus using 87 field-surveyed checkpoints. The checkpoints were surveyed using a combination of RTK GNSS and total station observations, with a least squares adjustment subsequently performed using a commercial software package.

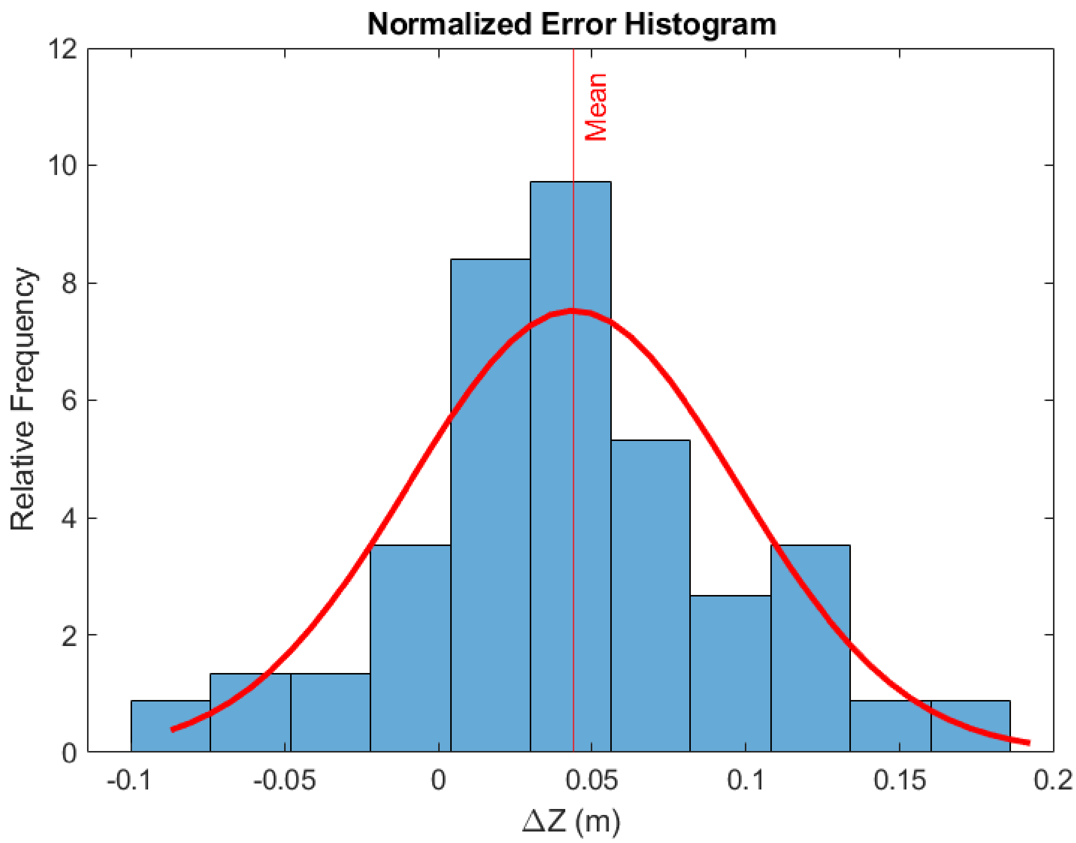

Figure I.B.1 Example of an error histogram. (Courtesy: Dr. Parrish lecture notes)
The orange curve is a fitted Gaussian distribution. The vertical line denotes the location of the mean. This histogram has been normalized, such that the area under the plot is equal to one.

Important items to look for in the visual test include:

1. The mean should be near zero, as a large (positive or negative) mean indicates the presence of bias in the data.
2. There should be no spikes far from the mean, as these would indicate the presence of outliers.
3. The distribution should be symmetric about the mean (not positively or negatively skewed).

4. The general shape of the error histogram should approximate the "bell-shaped curve" of the normal (Gaussian) distribution.

Following the visual assessment of the error histogram, the next step is to perform a quantitative normality test. The Lilliefors test for normality is recommended. The Lilliefors test is based on, but includes improvements to, the well-known Kolmogorov-Smirnov (K-S) test. Importantly, the Lilliefors test is available as a built-in function in commercially-available spreadsheet software packages and programming languages (Figure I.B.2). Another well-known and widely-used normality test is the Shapiro-Wilk (S-W) test.

```
%% Perform Lilliefors test for normality
h = lillietest(deltaZs);
if h == 0
    disp('DeltaZs PASS Lilliefors test for normality')
elseif h == 1
    disp('DeltaZs FAIL Lilliefors test for normality')
else
    disp('Warning: check format of input data')
end
```

Figure I.B.2 Lilliefors test for normality implemented in MATLAB. (Courtesy: Dr. Parrish lecture notes)

When analyzing the error histogram shown in Figure I.B.1, visual analysis confirms that the error distribution looks reasonable, although the mean of 4 cm indicates a positive bias (i.e., the lidar data are, on average, 4 cm too high with respect to the checkpoints), and the distribution is slightly positively skewed. Visual assessment indicates a lack of outliers. This error distribution passes Lilliefors test for normality. However, this test data set does not satisfy the criterion of mean error, μ < 25% of the RMSE, which is discussed in Section 7.2 of the ASPRS Positional Accuracy Standards for Digital Geospatial Data. In this case, the mean error is 64% of the RMSE, indicating that the RMSE is dominated by a large bias. This bias should be investigated further and removed if necessary, according to the instructions provided in Section C below.

B.2 Interpreting the Normality Test

There are many reasons why errors may not be normally distributed, and it is important to recognize that failing a normality test (the visual and/or quantitative portion) does not necessarily indicate a problem with the data, the checkpoints, or the test. However, assessing the results of the normality test can often help uncover mistakes or other issues made during testing.

For example, say a data producer finds that the errors for a particular data set fail a normality test. The data producer then conducts a follow-up investigation that reveals that incorrect boresight calibration parameters were applied while processing the data. In this hypothetical example, perhaps the original data met the required accuracy, as specified in the contract, but reprocessing the data with the correct boresight parameters applied leads to even better accuracy and normally-distributed errors.

Or, for another example, say assessment of the normality test results leads to the discovery of one or more checkpoints whose corresponding residuals exceed three standard deviations from the mean. While it is improper and in violation of these Standards to exclude checkpoints from the accuracy test simply because their corresponding errors are large (without additional justification), further analysis of the checkpoint or checkpoints in question may provide important insight. Perhaps, in a particular airborne lidar project, there was a two-week gap of time between the checkpoint survey and aerial survey, and it is discovered that a parking lot in which two of the checkpoints were located was repaved in this interval. The latter example illustrates the reason that the time between the data collection and the checkpoint survey should be minimized to the extent possible, with concurrent data collection being preferred, when feasible.

As one final example, it might be discovered that one of the checkpoints, which was surveyed with RTK GNSS, was near a tall chain-link fence, and subsequent analysis of the GNSS data may indicate that the checkpoint coordinates were affected by poor satellite geometry and multipath errors.

In all of these examples, if the accuracy test is repeated with these checkpoints withheld, the accuracy report must clearly state exactly which checkpoints were withheld, and it must provide a detailed justification.

B.3 Reporting the Normality Test

It is recommended that the error histogram be included and discussed in the accuracy report, so that it may provide context to the reported accuracy statistics. The accuracy report should state the type of normality test performed (e.g., the Lilliefors test or the Shapiro-Wilk test). If the error distribution fails the normality test (visual and/or quantitative portion), this should be stated and discussed in the report, including any findings from subsequent analysis as in the examples given above.

SECTION C: UNDERSTANDING ACCURACY STATISTICS

Contributor: Dr. Qassim Abdullah, Woolpert, Inc.

Section 5 of the ASPRS Standards provided brief definitions for the statistical terms computed during product accuracy assessment, as required by these Standards. These terms are Root-Mean-Square Error (RMSE), standard deviation (StDEV), median, and mean. In this section, we will focus on how to use the mean, standard deviation, and RMSE to investigate the presence of biases in the data; but, before we do that, let us elaborate on the statistical meaning of the standard deviation and the RMSE, as this will aid in analyzing the results and spotting biases when they occur.

C.1 Standard Deviation

Standard deviation is a statistical measure for the fluctuation or dispersion of individual errors around the mean value of all errors in dataset. The standard deviation is calculated as the square root of variance by determining each error's deviation relative to the mean, as given in the following equation and in Section D.1.1 of the ASPRS Standards:

$$s_x = \sqrt{\frac{1}{(n-1)}\sum_{i=1}^{n}(x_i - \overline{x})^2}$$

If error values fluctuate greatly farther away from the mean, we can expect the computed standard deviation to be high. Lower fluctuations close to the mean indicate a low standard deviation. Figure I.C.2 illustrates two accuracy assessment results. In case a, where the standard deviation is low, the fluctuation or spread of the errors from the mean is low, falling between 0.0 and 0.25 m for a range of 0.25 m. The second test, case b, illustrates data with a wide spread of errors which fluctuate around the mean at a range of 1.84 m, resulting in a high standard deviation value.

C.2 Root-Mean-Square Error (RMSE)

As defined in Section 5 of the ASPRS Standards, RMSE is the square root of the average of the set of squared differences between data set coordinate values and coordinate values from an independent source of higher accuracy for identical points. Thus, RMSE differs from standard deviation in that RMSE encompasses the biases that exist in the data. This becomes clear when you compare the equation for calculating the Root-Mean-Square Error to that of the standard deviation, given in Section D.1.1 of the ASPRS Standards. RMSE is calculated via the following equation:

$$RMSE_x = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(x_{i(map)} - x_{i(surveyed)})^2}$$

Note that we do not subtract the mean error, so RMSE represents the full spectrum of the error present in a checkpoint, including the mean error. However, in the equation for standard deviation, we subtract the mean error from every checkpoint error:

$$s_x = \sqrt{\frac{1}{(n-1)}\sum_{i=1}^{n}(x_i - \overline{x})^2}$$

This makes standard deviation a measure for the fluctuation of individual errors around the mean value of all errors.

C.3 Biases and Systematic Errors in Data

Geospatial mapping products are subject to systematic errors or biases from a variety of sources. These biases can be caused by things like using the wrong version of a datum during the product production process, or using the wrong instrument height for the tripod during the survey computations for the ground control points or the checkpoints. There are other sources of bias that can be introduced during the production processes. For instance, using the wrong elevation values in digital elevation data can result in biases during the orthorectification process. Also, using the wrong camera parameters (such as focal length) or the wrong lens distortion model can lead to biases in the final mapping product.

Systematic error can cause the product to fall below acceptable project accuracy levels. Thankfully, provided the appropriate methodologies are applied, systematic error can be identified, modeled, and removed from the data. This is not the case with random error: even if we discover it, we cannot eliminate it. However, we can minimize its magnitude through adherence to stringent production process and sound quality control practices. To illustrate systematic errors or biases in data, we will evaluate the scoreboards of four archers that vary in their aiming skills, illustrated in Figure I.C.1 below.

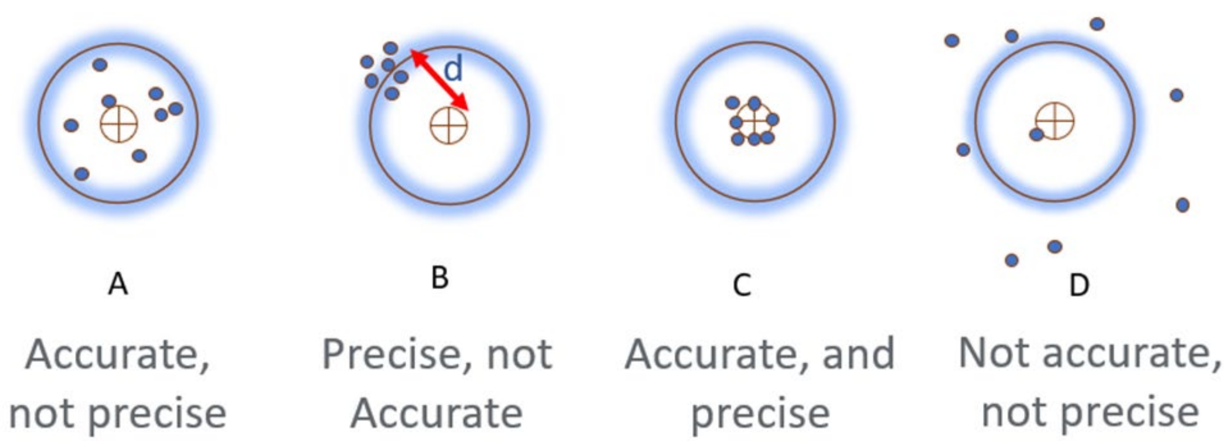

Figure I.C.1 Scoreboards for four archers with varied aiming skills. (Courtesy: Dr. Abdullah lecture notes)

For board A, the archer got the arrows around the bullseye, but the shots are scattered spatially around the center point. By contrast, board B reflects good spatial clustering, but the shots are clustered around a point far away from the bullseye. Board C is what you want your accuracy to be, with all shots clustered at the spot we aimed for. Board D demonstrates extremely undesirable results, possessing neither good clustering nor good aiming.

When we measure accuracy, results like boards B and C are the most desirable. Board C is preferred, as it represents clean results: all shots are at the bullseye. We can describe Archer C as “accurate and precise”. Although Archer B’s results lack good aim, the shots are clustered well; here we describe Archer B as “precise but not accurate”. Even though Archer B is not accurate, why are these results still acceptable? Examine the scoreboard for Archer B again: if we shift the locations of all the clustered arrows by a fixed distance *d*, the results will match the results from board C. This distance *d* represents the systematic error; once it is corrected, the final accuracy will be satisfactory.

But why did such a precise archer miss the bullseye to begin with? We must consider what may have taken place at the firing range to cause Archer B to miss. Perhaps the archer was using a sight scope hooked to his bow. Having all the arrows land in a tight cluster away from the bullseye is a strong indication of a mechanical failure of the sight scope that caused the arrows to go to the wrong place. Once Archer B’s sight scope is properly calibrated, his scoreboard in the second archery session will look just like Archer C’s board. The same logic can be applied to geospatial products like lidar point clouds or orthoimagery. That is why it is crucial to use accurate checkpoints when verifying product accuracy. It will help us quantify any existing systematic errors, allowing us to remove this error from the data in the same way that properly calibrating Archer B’s sight scope corrected his future shots.

C.4 Dealing with Biases

The best way to detect and evaluate biases is by analyzing the errors for each checkpoint, as well as analyzing the computed statistical terms required by the ASPRS Standards. It is particularly important to compare the mean value of the errors to the RMSE and StDEV values. For dataset with biases, most if not all reported errors shall have the same sign (positive or negative). Producing a graphical layout for these errors’ vectors will help reveal and diagnose biases in the data.

For a dataset with no biases, the RMSE and the StDEV will be very close, if not identical, to each other. Figure I.C.2 presents two datasets without any biases in them as the mean approaches zero. In such perfect cases of normal error distribution, you notice that the RMSE and the StDEV are equal in both datasets. Table I.C.1 lists the accuracy computation results for case a, illustrated in Figure I.C.2-a.

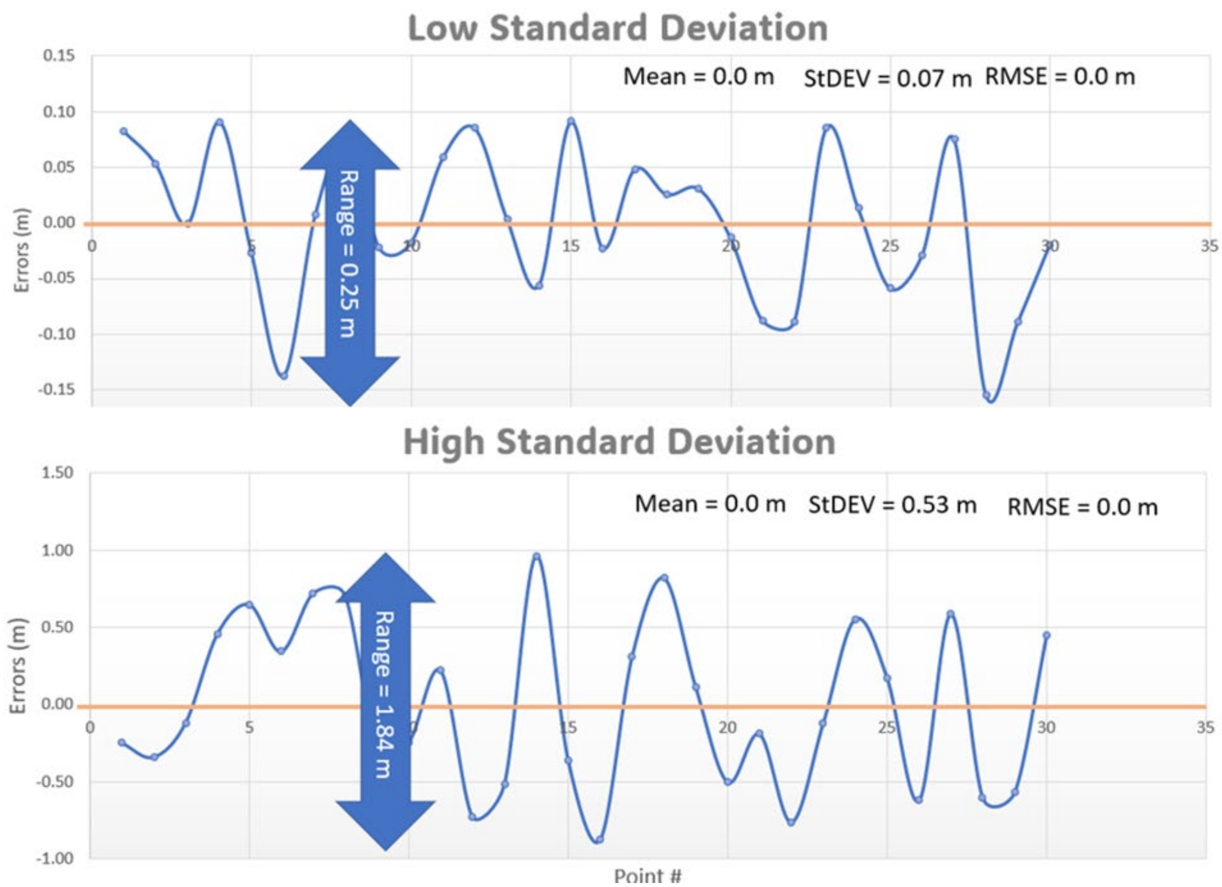

Figure I.C.2 Error representation and computed statistical terms in the absence of systematic errors. a (top): data with small standard deviation; b (bottom): data with high standard deviation. (Courtesy: Dr. Abdullah lecture notes)

Example on Bias Introduction in Data: Let us now assume a scenario in which systematic error was introduced into a lidar dataset during the product generation. Say a technician used the wrong version of the geoid model when converting the ellipsoidal heights of the point cloud to orthometric heights, which caused a systematic error or bias of 0.16 m in the computed elevation of the processed lidar point cloud. Figure I.C.3-a presents the same datasets as Figure I.C.2-a, but with a 0.16 m bias introduced into the elevation of the data.

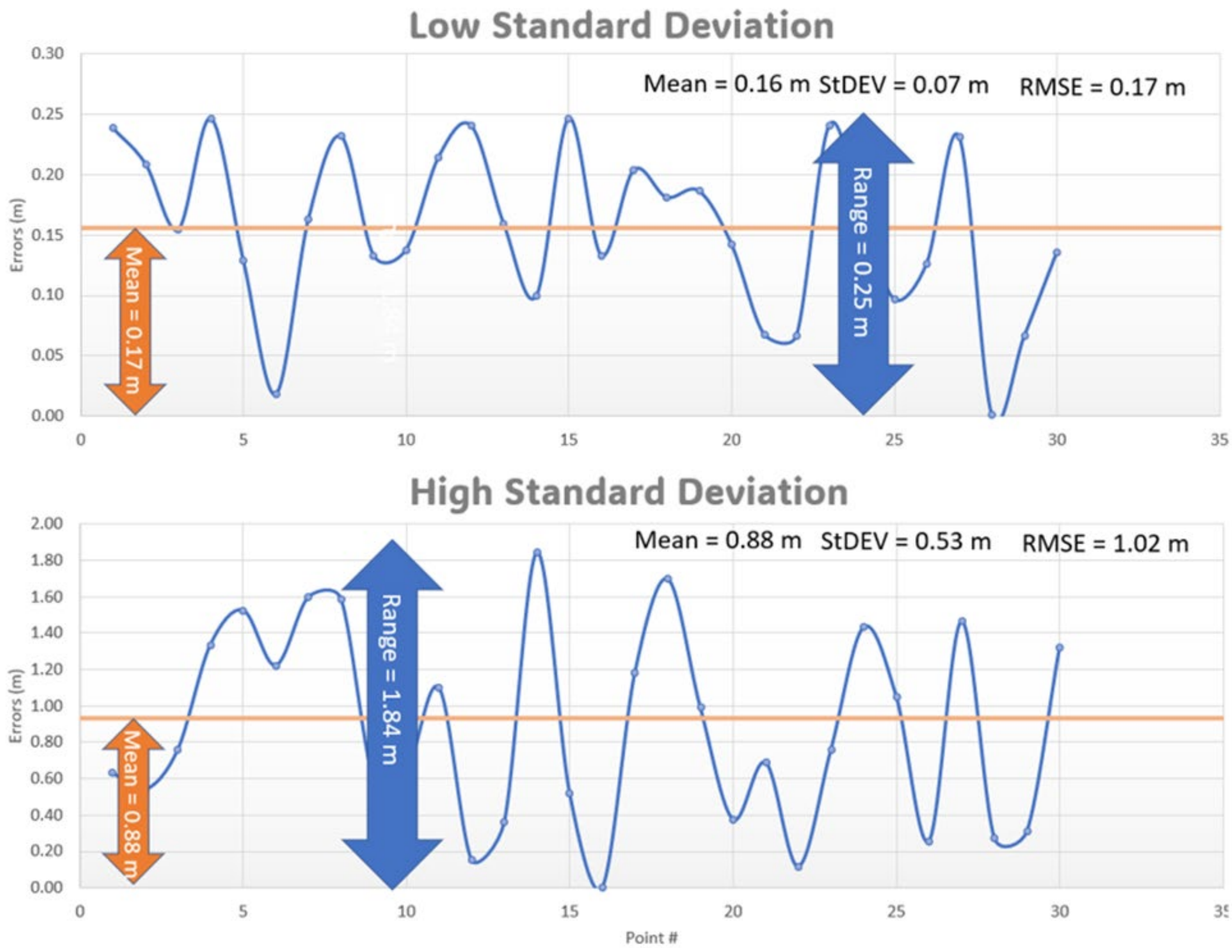

Figure I.C.3 Error representation and computed statistical terms in the presence of systematic errors. a (top): data with small standard deviation; b (bottom): data with high standard deviation. (Courtesy: Dr. Abdullah lecture notes)

Table I.C.2 lists the accuracy computations for a dataset that clearly inherited systematic errors, also illustrated in Figure I.C.3-a. Notice in Figure I.C.3, the errors do not fluctuate around the zero value anymore, but instead fluctuate around the elevated value of the mean at 0.16 m and 0.88 m, respectively.

Table I.C.1 Accuracy assessment table for an unbiased dataset.

Point #	Surveyed Coordinates			Lidar	Error Values (m)
	Easting (m)	Northing (m)	Elevation (m)	Elevation (m)	
CP_1	746093.605	97840.580	332.708	332.625	0.083
CP_2	746084.481	97875.486	333.856	333.802	0.053
CP_3	746076.993	97906.423	334.791	334.792	-0.001
CP_4	746069.043	97934.869	335.829	335.738	0.091
CP_5	746059.191	97968.525	336.837	336.864	-0.027
CP_6	746051.284	97996.814	337.671	337.808	-0.138
CP_7	746044.837	98025.039	338.717	338.709	0.007
CP_8	746036.494	98055.805	339.823	339.747	0.076
CP_9	746027.369	98082.550	340.646	340.669	-0.022
CP_10	746019.781	98112.192	341.636	341.654	-0.018
CP_11	746012.222	98144.373	342.792	342.733	0.059
CP_12	746006.094	98171.008	343.667	343.582	0.085
CP_13	745998.080	98196.380	344.486	344.482	0.004
CP_14	745987.766	98231.319	345.597	345.654	-0.056
CP_15	745939.681	98221.349	347.036	346.945	0.091
CP_16	745950.670	98190.848	345.788	345.811	-0.023
CP_17	745956.968	98166.660	344.999	344.951	0.048
CP_18	745966.818	98133.845	343.825	343.800	0.026
CP_19	745977.417	98100.689	342.676	342.645	0.031
CP_20	745986.146	98071.263	341.594	341.607	-0.013
CP_21	745994.431	98044.637	340.573	340.661	-0.088
CP_22	746003.437	98011.200	339.403	339.492	-0.089
CP_23	746013.675	97977.662	338.426	338.341	0.085
CP_24	746020.633	97952.708	337.451	337.438	0.013
CP_25	746029.450	97922.620	336.316	336.375	-0.059
CP_26	746037.820	97896.313	335.422	335.451	-0.029
CP_27	746073.182	98205.333	343.418	343.342	0.075
CP_28	746137.202	98304.228	344.254	344.409	-0.155
CP_29	746046.203	97866.550	334.320	334.409	-0.089
CP_30	746056.297	97832.573	333.199	333.219	-0.020
			Number of Checkpoints		30.000
			Minimum Error		-0.155
			Maximum Error		0.091
			Mean Error		0.000
			Median Error		0.001
			Standard Deviation		0.069
			RMSE		0.067
			Horizontal Positional Accuracy (E & N)		N/A
			Vertical Positional Accuracy		0.067
			3D Positional Accuracy		N/A

Table I.C.2 Accuracy assessment table for a biased dataset.

Point #	Surveyed Coordinates			Lidar	Error Values (m)
	Easting (m)	Northing (m)	Elevation (m)	Elevation (m)	
CP_1	746093.605	97840.580	332.708	332.469	0.239
CP_2	746084.481	97875.486	333.856	333.646	0.209
CP_3	746076.993	97906.423	334.791	334.636	0.155
CP_4	746069.043	97934.869	335.829	335.582	0.247
CP_5	746059.191	97968.525	336.837	336.708	0.129
CP_6	746051.284	97996.814	337.671	337.652	0.018
CP_7	746044.837	98025.039	338.717	338.553	0.163
CP_8	746036.494	98055.805	339.823	339.591	0.232
CP_9	746027.369	98082.550	340.646	340.513	0.134
CP_10	746019.781	98112.192	341.636	341.498	0.138
CP_11	746012.222	98144.373	342.792	342.577	0.215
CP_12	746006.094	98171.008	343.667	343.426	0.241
CP_13	745998.080	98196.380	344.486	344.326	0.160
CP_14	745987.766	98231.319	345.597	345.498	0.100
CP_15	745939.681	98221.349	347.036	346.789	0.247
CP_16	745950.670	98190.848	345.788	345.655	0.133
CP_17	745956.968	98166.660	344.999	344.795	0.204
CP_18	745966.818	98133.845	343.825	343.644	0.182
CP_19	745977.417	98100.689	342.676	342.489	0.187
CP_20	745986.146	98071.263	341.594	341.451	0.143
CP_21	745994.431	98044.637	340.573	340.505	0.068
CP_22	746003.437	98011.200	339.403	339.336	0.067
CP_23	746013.675	97977.662	338.426	338.185	0.241
CP_24	746020.633	97952.708	337.451	337.282	0.169
CP_25	746029.450	97922.620	336.316	336.219	0.097
CP_26	746037.820	97896.313	335.422	335.295	0.127
CP_27	746073.182	98205.333	343.418	343.186	0.231
CP_28	746137.202	98304.228	344.254	344.253	0.001
CP_29	746046.203	97866.550	334.320	334.253	0.067
CP_30	746056.297	97832.573	333.199	333.063	0.136
			Number of Checkpoints		30.000
			Minimum Error		0.001
			Maximum Error		0.247
			Mean Error		0.156
			Median Error		0.157
			Standard Deviation		0.069
			RMSE		0.170
			Horizontal Positional Accuracy (E & N)		N/A
			Vertical Positional Accuracy		0.170
			3D Positional Accuracy		N/A

C.5 Treating Biases in Data

Now, let us see how we are going to use the results presented in Tables I.C.1 and I.C.2 to assess the accuracy computations and whether we can spot problems in the data. Here is the suggested strategy:

First, look at the error mean values in both tables. We clearly notice that the mean error in Table I.C.2 is high, so we will focus on the results in Table I.C.2 for further analysis. A high mean error value is a good indication that biases are present in the data, but we need to further investigate how high the mean value is compared to RMSE and standard deviation. Slight differences between these statistical measures' values are acceptable.

Then, compare the RMSE and the standard deviation. Note that they are 0.069 m and 0.170 m, respectively. Having an RMSE value that is more than twice the standard deviation is a strong indication that biases may be present in the data. Remember, in the absence of systematic error (i.e., biases), the RMSE and the standard deviation should be equal. This conclusion is also supported by the fact that the mean is twice as high as the standard deviation.

Now that we have concluded that the data has biases in it, let us see how we will remove this bias without reproducing the product from scratch. For lidar data, we will need to raise or lower the computed heights for the point cloud by the amount of the bias—in this case, 0.16 m. Since the mean is a positive value, and the values in the "Error Values" column were computed by subtracting the lidar elevation from the checkpoint elevation, or:

Error = Surveyed Elevation – Lidar Elevation

we can then conclude that the terrain elevation as determined from the lidar data is lower than that measured by the surveyed checkpoints. Thus, we need to raise the lidar elevation by 0.16 m. Table I.C.3 illustrates the bias treatment we introduced above where the modified accuracy assessment values are listed in column "Unbiased Error Values". All we did here was raise, or "z-bump", the elevations of the point cloud by the amount of the bias, 0.16 m.

Similarly, if such an analysis were conducted to investigate the horizontal positional accuracy of an orthoimage, all we would need to do is modify the coordinates of the tile's header by the amount of the calculated biases without the need to reproduce the orthoimages. It is worth mentioning that removing the bias based on the "mean" value will not necessarily reduce the value of the RMSE by the same amount, as the degree of improvement in the recalculated RMSE value depends on the value of the standard deviation. For data sets with low standard deviation values and low rates of fluctuation, removal of the biases will improve the RMSE by a more significant degree.

Table I.C.3 Accuracy assessment table after bias removal.

Point #	Surveyed Coordinates			Biased Lidar	Unbiased Lidar	Unbiased Error Values (m)
	Easting (m)	Northing (m)	Elevation (m)	Elevation (m)	Elevation (m)	
CP_1	746093.605	97840.580	332.708	332.469	332.625	0.083
CP_2	746084.481	97875.486	333.856	333.646	333.802	0.053
CP_3	746076.993	97906.423	334.791	334.636	334.792	-0.001
CP_4	746069.043	97934.869	335.829	335.582	335.738	0.091
CP_5	746059.191	97968.525	336.837	336.708	336.864	-0.027
CP_6	746051.284	97996.814	337.671	337.652	337.808	-0.138
CP_7	746044.837	98025.039	338.717	338.553	338.709	0.007
CP_8	746036.494	98055.805	339.823	339.591	339.747	0.076
CP_9	746027.369	98082.550	340.646	340.513	340.669	-0.022
CP_10	746019.781	98112.192	341.636	341.498	341.654	-0.018
CP_11	746012.222	98144.373	342.792	342.577	342.733	0.059
CP_12	746006.094	98171.008	343.667	343.426	343.582	0.085
CP_13	745998.080	98196.380	344.486	344.326	344.482	0.004
CP_14	745987.766	98231.319	345.597	345.498	345.654	-0.056
CP_15	745939.681	98221.349	347.036	346.789	346.945	0.091
CP_16	745950.670	98190.848	345.788	345.655	345.811	-0.023
CP_17	745956.968	98166.660	344.999	344.795	344.951	0.048
CP_18	745966.818	98133.845	343.825	343.644	343.800	0.026
CP_19	745977.417	98100.689	342.676	342.489	342.645	0.031
CP_20	745986.146	98071.263	341.594	341.451	341.607	-0.013
CP_21	745994.431	98044.637	340.573	340.505	340.661	-0.088
CP_22	746003.437	98011.200	339.403	339.336	339.492	-0.089
CP_23	746013.675	97977.662	338.426	338.185	338.341	0.085
CP_24	746020.633	97952.708	337.451	337.282	337.438	0.013
CP_25	746029.450	97922.620	336.316	336.219	336.375	-0.059
CP_26	746037.820	97896.313	335.422	335.295	335.451	-0.029
CP_27	746073.182	98205.333	343.418	343.186	343.342	0.075
CP_28	746137.202	98304.228	344.254	344.253	344.409	-0.155
CP_29	746046.203	97866.550	334.320	334.253	334.409	-0.089
CP_30	746056.297	97832.573	333.199	333.063	333.219	-0.020
					Number of Checkpoints	30.000
					Minimum Error	-0.155
					Maximum Error	0.091
					Mean Error	0.000
					Median Error	0.001
					Standard Deviation	0.069
					RMSE	0.067
					Horizontal Positional Accuracy (E & N)	N/A
					Vertical Positional Accuracy	0.067
					3D Positional Accuracy	N/A

SECTION D: LIDAR DATA QUALITY VERSUS POSITIONAL ACCURACY

Contributor: Dr. Qassim Abdullah, Woolpert, Inc.

When modeling terrain with lidar, it is important to be aware of the difference between elevation data quality and positional accuracy. In many instances, users of lidar data focus solely on point cloud accuracy as specified by sensor manufacturers, but an accurate lidar point cloud does not necessarily result in accurate modeling of the terrain, nor will it create accurate volumetric calculations: elevation data must also faithfully represent the terrain detail. Therefore, users should also consider point density as it relates to terrain roughness or smoothness, as this is an equally important aspect of accurate terrain modeling.

Terrain modeling methodologies (e.g., polygon-based Regular Triangulated Networks (RTNs) versus Triangulated Irregular Networks (TINs) versus Voxel-Based Networks) also affect the terrain model quality. Terrain analysis is sensitive to whether the software represents the point cloud as a TIN, a gridded surface, or an RTN. Methods that involve gridding the data are sensitive to grid cell size (post spacing). Note that lidar point density is an important factor when choosing grid cell size.

Figure I.D.1 illustrates the relationship between terrain roughness and point density. While the point cloud in this example may have a vertical accuracy of $RMSE_V$ = 10-cm, TIN interpolation based on surrounding areas of low point density places the vertical position of point A at point A’, resulting in a vertical error of 2 meters in this example. The remedy is to obtain the point cloud at a higher density so that it more accurately represents the terrain detail. Attempting to use a low-density point cloud to represent terrain with high frequencies of undulation will result in inaccurate volume estimations, regardless of what software or modeling algorithms are used. Smoother terrain may be adequately represented with a lower density point cloud. Very smooth or flat terrain can be accurately modeled using a point cloud with nominal post spacing (NPS) of a few meters or coarser.

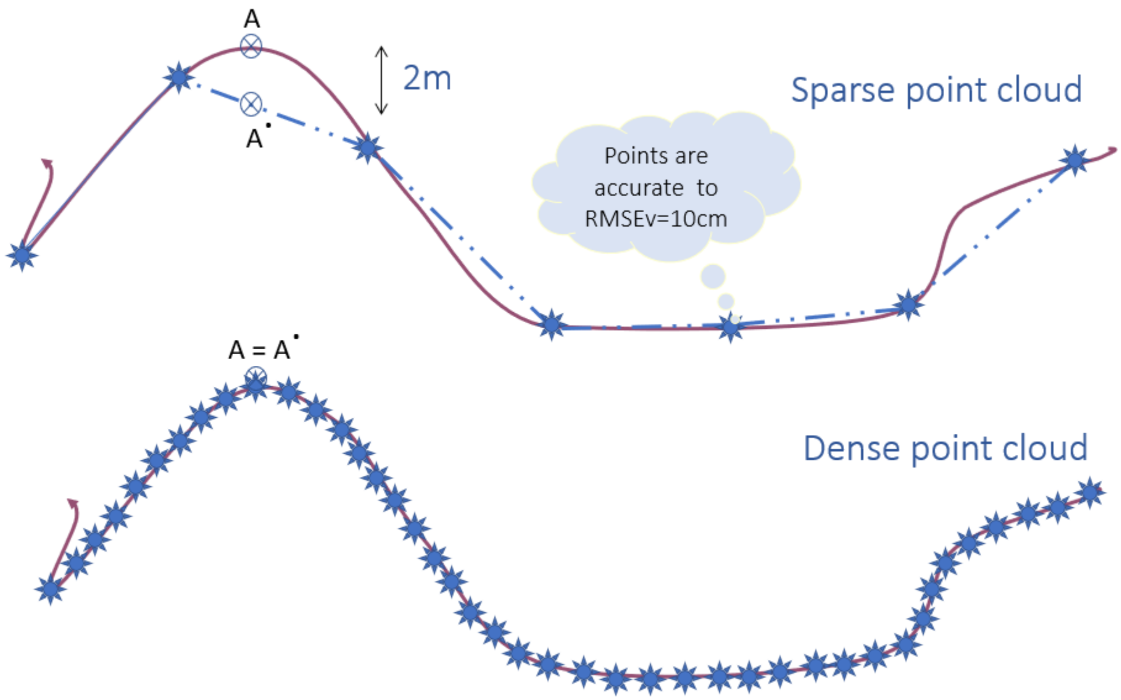

Figure I.D.1 Terrain Model Quality as a Function of Point Density and Vertical Accuracy (Courtesy: Dr. Abdullah lecture notes)

The Nyquist-Shannon sampling theorem, which is well-known and widely used in signal processing, may be used to determine the point density required to accurately represent the project terrain. According to the Nyquist-Shannon sampling theorem, if a signal x(t) contains no frequencies higher than B Hz, then a sampling rate of greater than 2B samples per second (or 2B Hz) will be needed in order to reconstruct the original signal without aliasing.

For example, let us assume that the undulation rate of the terrain represents the highest frequency of the signal to be modeled, and the nominal point spacing represents the sampling rate needed to model the terrain without aliasing. If we want to accurately model rocky terrain where the spikes caused by these rocks appear every 30 cm on average, the nominal point spacing of the lidar data used to model this terrain should be less than 15 cm.

SECTION E: LIDAR SYSTEM CLASSIFICATION AND GROUPING

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E.1 Introduction

Mapping professionals new to lidar technology generally find themselves faced with the challenge of selecting the right lidar system for their needs from the many systems available on the market today.

Due to the wide range of price and performance across various key subsystems (laser rangefinders, scanner types, inertial navigation and positioning systems, etc.), lidar systems may vary greatly from each other. Specific sensor designs may not fit easily into a category, or they may straddle different group definitions or technical performance envelopes. This section is intended to provide a broad overview of the major technical differences between designs for professionals who are unfamiliar with lidar technology, but who may be considering incorporating it into their projects in the future.

E.2 Lidar System Performance

The overall performance of a UAS-based lidar system is determined by the technical specifications of the core subsystems, most critically the laser rangefinder (the lidar subsystem), the scanner, and the navigation subsystem (which is responsible for the position and orientation of the sensor during flight). As with any complex system design, overall achievable accuracy is determined by the total combined error budget of all subsystems. When optimizing the price/performance curve for a sensor design, it is important to match the performance of each subsystem against the component price and against each other. A low-end IMU may not be able to make use of a high-performing lidar component with exceptional range performance and very low shot noise. Similarly, using the highest accuracy positioning solution available on the market may not improve the inherent limitations of using low-cost, low-spec lidar. When considering the design of the UAS lidar system, care should be taken to ensure subsystems are not mismatched in terms of technical specifications. Technical specifications regarding achievable performance should always be based on the overall system performance, not on the individual specifications of the various subsystems.

E.3 Lidar System Classification

Since lidar technology is still evolving, the geospatial mapping industry has not yet developed a mature classification system to categorize the different lidar systems available on the market. The following group designation system is based around payload weight and/or primary platforms used and is only intended to function as a general guide. Please note that some sensor designs may not easily fit into a specific grouping.

Group 0 – Consumer-Grade Lidar

Consumer products such as smartphones are starting to include lidar scanning as a function alongside their built-in digital cameras. Lower-cost, lower-performance “2D” lidars or basic rangefinders are becoming more common in robotic vision applications such as autonomous vacuum cleaners. While the embedded lidar technology in consumer products is improving, such consumer-grade lidar systems are not considered suitable for professional aerial mapping applications.

Group 1 – UAS-Mounted Lidar < 2.5 kg Payloads

Starting in approximately 2010, the automotive industry has created a demand for compact, lower-priced UAS lidar systems for use in self-driving vehicles. The size, weight, and power of these compact lidar systems, when paired with lower-cost position and orientation systems, makes them suitable for many remote sensing applications, such as aerial mapping using a UAS, mobile mapping using a vehicle, or personal/on-foot mapping using a backpack. The range of options in this group is varied, covering everything from older opto-mechanical designs with spinning mirrors to newer solid-state designs with no moving parts. These lidars benefit from their compact size, low power requirements, and lower cost compared to more traditional lidar systems. Due to the range of options in this group, generalizations are difficult to make, but these systems tend to have all or most of the following characteristics:

- Single wavelength operation in the Near-Infrared (NIR) around 900 nm.
- Multi-channel designs typically featuring 8x, 16x or 32x individual laser transmitters. Designs with 64x or 128x channels are less common but may be available.
- 360-degree rotating mirrors for scanning with a fixed forward/backward spread of 10–20 degrees due to the multiple channels (a fan of beams).
- Fixed fields of view for solid-state imagers.
- Range performance to 20% reflectivity targets is typically in the 50–300 m range, due to the lower peak pulse power and reduced sensitivity of the receiver designs compared to lidars designed for higher-altitude operations.
- Detectability (percent of pulses return a specified range to specified reflectivity target) better than 50%, with most newer designs rating better than 90%.
- Asymmetrical beams with divergences in the 1–3 milliradian (or higher) range and with a major and minor axis, resulting in an elliptical beam pattern on the ground.
- Higher shot-to-shot noise (higher range error), resulting in greater peak-to-peak noise or "fuzziness" on hard surfaces.
- Multiple returns on most, but not all, systems, with most systems capturing 2–5 returns per pulse.
- Intensity (return pulse amplitude) captured as 8- or 12-bit values, typically not normalized to pulse energy.
- Traditional time-of-flight (ToF) return pulse detection method.
- Achievable vertical accuracy ($RMSE_z$) compared to control (network accuracy) better than 10 cm, with most systems capable of better than 5 cm given a good GNSS position solution and a compatible IMU for orientation.
- Precision of 3.0–7.0 cm over hard surfaces (single pass, 1σ deviation) with peak-to-peak noise of 10–20 cm over the same surface (before any smoothing of the point cloud).
- Integration with a compatible position and orientation system (POS) that provides approximate post-processed position accuracy of 0.02–0.05 m, 0.08° heading, 0.025° pitch/roll.

Group 2 – UAS-Mounted Lidar < 10.0 kg Payloads

Moving up in price/performance from Group 1 typically involves moving to systems integrated with lidars expressly built for long range mapping applications and paired with higher-accuracy IMUs for better angular (orientation) measurements. This typically increases the payload weight and power requirements, requiring a drone with a greater lift capacity than would typically be used in Group 1 systems. There are a range of medium-lift commercial drones capable of carrying payloads up to 10 kg for extended durations, making them very suitable for mapping applications. Positional accuracy via GNSS tends to remain the same as for the prior group. With longer-range performance, the highest performing systems in this group are also capable of operating from fixed-wing or helicopter platforms flying at low altitudes. These systems are characterized by all or some of the following characteristics, relative to Group 1:

- Single wavelength operation on most systems in the 1.0–1.5-micron range. Dual (IR/Green) wavelength options available in some models for bathymetric applications.
- Range performance to 20% reflectivity targets is typically in the 100–1000 m range, due to higher peak pulse power from the laser and more sensitive receiver designs.
- Detectability (percent of pulses return a specified range to specified reflectivity target) better than 95%, with most newer designs reaching better than 99%.
- Improved beam quality with symmetrical beam shapes and beam divergences in the 0.3–0.7 milliradian range.
- Single-channel (single transmitted beam) design for many systems, though dual and triple beam designs are becoming more common. Some quad-channel designs are available for mobile mapping applications.
- Traditional time-of-flight (ToF) return pulse or full waveform digitization detection methods.
- Geiger-mode and single-photon receiver designs may be available in some models.
- Multiple returns, up to 15 in some designs. Last-pulse and multiple last-pulse logic is standard.
- Intensity captured up to 16-bit values, on some designs normalized to outgoing pulse energy to provide more uniform reflectance measurements.
- 360-degree rotating mirrors or oscillating mirrors (side-to-side) for scanning. Oscillating mirrors provide greater sampling rates for the same PRR by keeping all pulses in the effective field of view.
- Achievable vertical accuracy ($RMSE_z$) compared to control (network accuracy) better than 5-cm, with most systems capable of better than 2.5-cm if provided with a good GNSS position solution and a compatible IMU for orientation.
- Precision of 0.5–1.5 cm over hard surfaces (single pass, 1σ deviation) with peak-to-peak noise of 5–10 cm (before any smoothing of the point cloud).
- Integrated with a compatible position and orientation system (POS) that provides approximate post-processed position accuracy of 0.02–0.05 m, 0.035° heading, 0.015° pitch/roll.

Group 3 – Manned Aircraft Mounted Lidar

This group covers traditional aerial lidar systems deployed on fixed-wing and helicopter platforms. These are the established airborne lidar sensor designs that have been available commercially for 30 years. Built around time-of-flight measurements with purpose-built lidars and scanners, they use high-accuracy positioning and orientation subsystems to allow accurate mapping from ranges of 3000 m or more. They are optimized for wide area collection and high-density corridor mapping from fixed-wing survey aircraft or helicopter platforms. In addition to time-of-flight, Geiger-mode and single-photon designs are also utilized to achieve the greatest efficiency possible when mapping large regions. This group also includes most bathymetric lidar systems.

Group 4 – SLAM-Based Mapping Systems (Various Platforms)

Simultaneous Localization and Mapping (SLAM) sensor designs have become common in mapping applications over the past two decades. SLAM is a technique used in vision systems to allow a sensor to simultaneously map its surroundings in 3D space and locate itself accurately in that space. SLAM is used for real-time mapping, path/route planning, and obstacle avoidance in various applications.

SLAM is a general term, and there are a variety of different algorithms and implementation approaches that can all be considered “SLAM” systems. The term “visual SLAM” refers to the use of SLAM with imagery from on-board cameras, while “lidar (or laser) SLAM” refers to the use of lidar as the primary on-board sensor feeding the SLAM algorithm. Visual SLAM is more common in consumer appliances such as robotic vacuums, or in controlled environments such as warehouses, while lidar SLAM is more commonly used for mapping applications. The commercial development of SLAM sensors has been driven by the creation of efficient SLAM algorithms and the falling costs of key sensors such as digital cameras and lidars, along with the overall increase in on-board computing power available to process the SLAM algorithms using the sensor data in real-time.

A common characteristic of all SLAM-based mapping systems is that they work without needing a GNSS positioning solution, unlike traditional mapping lidars and UAS lidar systems, allowing SLAM-based mapping systems to work in GNSS-denied environments such as underground areas, indoors, or in confined spaces. Instead, positioning is provided by identifying and tracking targets and landmarks in the local environment as the system moves through the space. Tying landmarks to known locations in a spatial reference frame allows for absolute positioning of the resulting data. With enough identifiable targets in the scene, achievable positional accuracy is similar to a post-processed kinematic (PPK) survey-grade L1/L2 GNSS solution. For mapping purposes, this can generally be taken as 5 cm or better. Note that both SLAM and GNSS have requirements that mirror each other; GNSS systems need a good view of satellites in the sky, while SLAM systems need a good view of 3D features or set targets on the ground. Both will degrade in positional accuracy and ultimately fail if their respective requirements are not met. Hybrid sensor designs using both GNSS and SLAM positioning are being investigated, but these are not generally available commercially as of 2024.

Since a SLAM system maps its environment as it moves, it can be used to map any 3D space that has suitable geometries for landmark and target identification. SLAM-based systems are being used more and more often to map outdoor spaces, sometimes in conjunction with UAS lidar mapping. However, there are limitations to mapping with SLAM sensors:

- SLAM systems need a feature-rich environment to identify tracking targets and landmarks for localization, so they may have issues in excessively monotonous environments, such as the top of a continuous canopy or long stretches of empty corridors. Additional control targets may need to be added for effective mapping in such environments.
- Scaling to map large outdoor areas is less efficient with SLAM than a traditional GNSS approach. SLAM collection efficiency usually decreases with area, while GNSS collection efficiency usually increases.
- Localization errors may accumulate over time. Appropriate steps should be taken in the algorithm and data processing to minimize loop closure problems.

For indoor (GNSS-denied, confined spaces) mapping projects, SLAM-based sensors currently offer the best approach for rapid, accurate mapping data collection. For outdoor (open-sky) mapping projects, factors such as the availability of suitable identifiable targets and landmarks in the area, the efficiency of collection, and the field logistics will affect how effective a SLAM-based mapping system will be compared to a GNSS system.

E.4 Lidar System Cost

There is a competitive commercial market for the sale and support of UAS lidar systems, with multiple vendors for both GNSS-based and SLAM-based systems designed for mapping applications. System integrators typically provide an integrated payload with a lidar, GNSS/IMU position and orientation subsystem (or a SLAM-based design), on-board processor, data recorder, flight planning software, and data post-processing software. The key subsystems can be sourced from third-party vendors, who may also offer integrated mapping systems themselves (e.g., Riegl, Teledyne, Hexagon) or who may only sell components to integrators (e.g., Hesai, Velodyne, Livox). It is increasingly common for drone manufacturers to offer integrated lidar payloads (e.g., DJI with the L1); alternately, they may partner with a lidar system integrator to offer an out-of-the-box lidar mapping solution. Most payload designs from system integrators offer an industry standard mounting option, such as a gimbaled or on-rail configuration, so the payload and the drone platform can be chosen independently of each other.

Due to the competitive market and the overall dynamic technology in the core subsystems (compact lidar system performance continues to improve significantly every 12–18 months), it is difficult to provide accurate and up-to-date price guidance for mapping professionals who may be considering acquiring such a system. A detailed price vs. performance analysis covering the major technical aspects of the system including achievable accuracy, ranging precision, effective operating range, canopy penetration effectiveness, and size/weight considerations should be conducted prior to any purchasing decision.

ADDENDUM II: BEST PRACTICES AND GUIDELINES FOR FIELD SURVEYING OF GROUND CONTROL POINTS AND CHECKPOINTS

Contents

TABLES

PURPOSE

The purpose of this Addendum is to provide best practices for the field surveying of Ground Control Points (GCPs) and Checkpoints, as referred to throughout these Standards. These guidelines are intentionally sensor and manufacturer agnostic. These best practices are not intended to replace the manufacturer's manual, nor do they replace surveying textbooks. Best practices recommended herein assume that the equipment operator and data processor understand surveying fundamentals and can competently operate the relevant equipment and software. These guidelines are not intended to instruct beginners in the performance of surveying tasks; rather, they represent a consensus reached by experienced professionals, and are intended to provide recommendations for seasoned surveyors.

SCOPE

Five methodologies for field surveying of ground control points (GCPs) and Checkpoints are covered in this Addendum:

1. Establishment of static control and best practices for utilizing Global Navigation Satellite System (GNSS) for Real-Time Kinematic (RTK) surveying using base and rover methodologies.
2. Use of GNSS Real-Time Networks (RTN).
3. Use of GNSS Real-Time Precise Point Positioning (RT-PPP) techniques to establish ground control in clear open-sky areas only.
4. Use of conventional surveying techniques (total stations) to establish Vegetated Vertical Accuracy (VVA) checkpoints under tree canopies, incorporating RTK/RTN techniques for local control.
5. Use of terrestrial scanning and mobile mapping methodologies to establish ground control points (GCPs) and Non-Vegetated Vertical Accuracy (NVA) checkpoints under controlled circumstances.

CONTRIBUTORS

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CAUTION

The surveying products described in this Addendum should only be provided by a competent and knowledgeable land surveyor who is familiar with the associated equipment, technology, and software. To ensure that survey projects meet these Standards, it is absolutely necessary that the surveyor possesses an appropriate understanding of error theory, practical surveying methodologies, and modern survey technologies such as survey-grade GNSS, geodesy, and map projections.

Additionally, even in jurisdictions that do not legally require that a licensed land surveyor oversee ground control surveying, it is recommended that the survey work described herein be supervised and certified by a professional land surveyor licensed in any one of the United States, as these professionals possess the requisite skill set to oversee survey projects and verify the accuracy of control points and collected data.

CERTIFICATION:

The following statement is an example of the certification which should accompany the delivery of the ground control points and checkpoints:

I, _____(FULL NAME) _____________do hereby certify that the ground control points (GCPs) and checkpoints provided herein meet the requirements as promulgated in the ASPRS Positional Accuracy Standards for Digital Geospatial Data Edition 2, Version 2 (2024), and that the coordinate values and the Coordinate Quality values are true and correct to the best of my knowledge and belief.

______________Signature & Professional Land Surveyor Designation________, Date____________

SECTION A: COORDINATE QUALITY OF CONTROL POINTS AND CHECKPOINTS

These guidelines are intended to assist field surveyors and Quality Assurance/Quality Control (QA/QC) personnel in the determination of the reliability of field measurements intended to control another type of geospatial data (e.g., aerial).

Because coordinate quality estimates are collected in different ways and represented by different statistical means depending on the equipment or software manufacturer, the professional surveyor must understand the coordinate quality definition particular to the software being used. According to these Standards, each control point and checkpoint collected in the field should have a horizontal and vertical accuracy of two times better than that of the aerial or other geospatial data being controlled and verified. Therefore, professional surveyors should ensure that their equipment, software, and field survey practices will allow the collected data to meet these requirements. It is always the professional surveyor’s responsibility to ensure this is performed correctly.

In the latest version of these Standards, the accuracy of the checkpoints themselves is included in the overall accuracy calculation of the lidar data. In order to reliably estimate the quality of the positions of these surveyed points**,** multiple independent observations of each surveyed point must be made. The recommended process to accomplish this when using GNSS is as follows:

1. The first observation on a point should be made for a minimum of 180 one-second epochs. This time frame is sufficient under ideal, clear sky conditions. If the sky view conditions are less than

ideal, a longer observation time is warranted, which could be 300 or more epochs depending on conditions.

2. Then, the GNSS antenna pole should be rotated 120 degrees, and the same survey process should be repeated.
3. After that is accomplished, the pole should be rotated another 120 degrees and a third, similar measurement should be taken.
4. If each of the three independent measurements do not agree within 20 mm in 3D (in other words, the 3D residual of each independent measurement exceeds 20 mm from the weighted mean, if available, or a simple average if not), then another measurement should be taken, again rotating the pole 120 degrees, until you have at least three measurements which agree to better than 20 mm. The best estimate of the correct position of the point is a weighted mean of all acceptable measurements taken, which should be a minimum of three measurements—possibly more in difficult conditions.
5. It is acknowledged that in some cases the stated Quality Level of a project may not require such rigorous methodology, although in more and more cases today it does. No matter what the Quality Level of the project, the surveyor should always provide some degree of redundancy in the survey of ground control and checkpoints to hopefully eliminate blunders or other large errors.

Additionally, coordinate quality indicators must be provided in a standardized format. One value should be provided for each of the 1D (or vertical), 2D (or horizontal), and 3D dimensions, at a one-σ level of confidence. These values should be included with all coordinate data. This information must be provided in any coordinate listing and/or survey report, as in the examples listed in Table II.A.1:

The preferred method of reporting is Root-Mean-Square Error or RMSE, to be consistent with the accuracy estimates of the lidar data. RMSE can be defined as the difference between true or predicted values and observed values or an estimator. Usually, RMSE requires a known value, to which an observed value may be compared. The residuals or differences between the known and observed values can be calculated, then squared, then summed, and finally divided by the number of observations. The square root of that numeric result is the RMSE of the data set. These calculations can be easily obtained in an Excel spreadsheet. A formula to find RMSE follows:

$RMSE = \sqrt{\frac{\Sigma(P_i - O_i)^2}{n}}$ where:

- Σ is a symbol that means "sum"
- P_i is the predicted or true value for the i^{th} observation in the dataset
- O_i is the observed value for the i^{th} observation in the dataset
- n is the sample size

If RMSE is not available from the hardware/software manufacturer, some software manufacturers provide Coordinate Quality (CQ) values which estimate accuracy rather than precision. Again, at least three independent measurements of the positional coordinates must be employed when calculating the CQ values.

The least desirable coordinate quality indicator is standard deviation, as it generally gives overly optimistic estimates of the reliability of the coordinates. Once more, in order to provide more reliable estimates at least three independent measurements must be taken and combined, as per steps one through four above.

If we do not have a known value to which to compare our measurements, we can use the weighted mean of the observations as a substitute, which will give us the Standard Deviation, which is considered a measure of precision rather than accuracy:

$$\sigma = \sqrt{\frac{\sum(x_i - \mu)^2}{N}}$$

To calculate the standard deviation of a set of numbers, we can use the above formula, where σ is the standard deviation, x_i is the i^{th} number in the set, μ is the mean of the set, and N is the number of elements in the set.

The weakness here is that if there are biases (errors) in the observations caused by something like an incorrect antenna height or incorrect coordinates at the RTK base, then the derived coordinates will be in error. This is where good survey practices and an understanding or error theory come in to help the competent surveyor eliminate such biases.

For a more detailed examination of accuracy statistics and of how to deal with biases in your survey data, please read Addendum I, Section C: Understanding Accuracy Statistics.

Table II.A.1 Survey Coordinate Samples (All values shown in the table are expressed in meters.)

Point ID	Northing	Easting	Ortho Height	Horizontal or 2D RMSE	Vertical or 1D RMSE	3D RMSE
G0001	496353.356	5941936.542	832.743	0.009	0.012	0.015

Point ID	Northing	Easting	Ortho Height	Hz Accuracy or 2D CQ	Vert Accuracy or 1D CQ	3D Accuracy or 3D CQ
G0137	396353.356	3941936.542	1832.743	0.008	0.013	0.015

Point ID	Northing	Easting	Ortho Height	Hz or 2D Std Dev	Vert or 1D Std Dev	3D Std Dev
N0108	460624.421	2641766.062	1083.664	0.007	0.011	0.013

The units of measure used must be indicated in the report or in the table.

In summary, the following table provides an estimate of the possible accuracies achievable using various field survey methodologies. These accuracies assume that observations are conducted under excellent observing conditions: clear sky, multiple GNSS constellations, reasonably short baselines, and no errors caused by blunders such as wrong rod or antenna heights or less than perfect starting coordinates.

WARNING: Absolute accuracy values should always be used to quantify the quality of ground control and checkpoints. The accuracy or precision estimates of these coordinates and how they are derived

vary widely amongst the different hardware and software manufacturers. As stated previously, standard deviations are often overly optimistic and may not identify biases in the data. It is up to the professional surveyor in charge of the ground survey to ensure that the survey data and its quality estimates are correct. In a situation where the surveyor is not confident that these estimates are reliable, the surveyor should use the predicted accuracies in Table II.A.2.

Table II.A.3 Predicted Accuracies

Survey Methodology		Predicted Accuracy Values	
	Horizontal	Vertical	3D
Adjusted Closed Loop – Digital Leveling		5 mm	
Real-Time Network Following Section C – Recommended Procedures	10 mm	16 mm	19 mm
Real-Time PPP After Convergence Following Section D – Recommended Procedures	15 mm	24 mm	28 mm
Real-Time Kinematic (RTK) Base and Rover Following Section B – Recommended Procedures	20 mm	32 mm	38 mm
Closed Conventional Traverse Following Section E – Recommended Procedures	25 mm	40 mm	47 mm
Real-Time PPP After Convergence, Single Measurement	20 mm	50 mm	54 mm

SECTION B: STATIC CONTROL AND RTK SURVEYING

The purpose of this section is to set forth best practices and guidelines for the establishment of GNSS static control points and the use of GNSS RTK using base and rover methodologies. Furthermore, these guidelines will provide practical suggestions for obtaining consistent and accurate three-dimensional survey control.

B.1 Static Surveying with GNSS

Static surveying requires that two or more GNSS receivers occupy stations at the ends of baselines for a time period determined according to baseline lengths, satellite constellations, the potential for satellite signal interference, multipath, and/or blockage caused by trees, buildings, etc. Most manufacturers quote the recommended observation times and accuracy specifications based on ideal conditions, but survey points are frequently located in less-than-ideal GNSS environments. It is always advisable to err on the side of caution by extending the duration of occupations, to avoid having to go back and resurvey due to poor data.

While it is possible to use only two receivers for a static survey, multiple receivers configured in a network using multiple known and unknown points will give better results. At least one receiver must occupy a point with precise, known coordinates; other receivers may be set up at unknown points. Raw data should be collected at all receivers simultaneously for a predetermined amount of time, which, after the baselines are put through post-processing, will produce the coordinates of the unknown points.

This survey method is most often used to perform precise positioning over large areas where baseline lengths are greater than real-time observations allow. The accuracy of the post-processed positions will be affected by the length of the various baselines, the observation duration of the survey, the precision of the equipment mounting system used, the number of independent redundant observations, and the accuracy of the existing control points used to constrain the surveyed data.

B.1.1 Equipment

Numerous makes, models, and combinations of GNSS antennas, GNSS receivers, and data controllers are available on the current market and are continuously evolving. Usually, it is easiest to conduct surveys with antennas, receivers, and mounting equipment (tripods, tribrachs, and height measuring devices) from the same equipment manufacturer. However, surveyors should make equipment choices based on best practices, desired accuracy results, and project requirements. Be mindful that processing software and workflows can be manufacturer dependent.

B.1.2 Data Management

When planning static surveys, be mindful of data size and storage requirements, as well as pre-project observation planning. Consider the following when planning static surveys:

File Size and Storage Capacity: Consider the size and duration of your files and where the data will be logged. There must be enough free memory storage in the receiver for the desired survey. This is not usually a concern with modern GNSS receivers, which possess much larger storage capacities than their predecessors, but it can become an issue with some older models.

Data Sample Rate: For static surveys, it is common to collect data at a 5- to 15-second sampling interval, rather than at a 1-second interval. Data should be collected at the same interval for all receivers where possible—some Continuously Operating Reference Stations (CORS) will collect and disseminate data at a minimum of a 30-second rate. Sample rate will directly affect the file size.

Elevation Mask/Cutoff Angle: Consider the environment and obstructions that may potentially block GNSS signals. Above what elevation mask do you want the receiver to log raw data? The commonly recommended value is 15 degrees. Remember that a low elevation mask will allow the collection of poorer-quality raw data due to signal blockage, multipath, atmospheric interference, etc. The use of a higher elevation mask will eliminate much of the poorer data, and will almost always improve the vertical component of the solution. The most advanced processing software can mitigate most of the effects of low-quality raw data, but not everyone uses the same software. Therefore, it is better to err on the side of caution rather than to have to go back and re-observe due to poor-quality raw data.

B.1.3 Workflow

Initial site analysis and pre-project planning are essential to project success; therefore, it is essential to plan missions, pre-determine locations that possess adequate sky coverage, and minimize potential multi-path contributors. Always consider the environment being surveyed, and account for potential obstacles such as vegetation, structures, canyon walls, or any other objects that may obstruct your receiver's view of the satellites.

B.1.4 Preparation

- Plan for power needs. Bring adequate chargers and batteries.
- Plan for memory needs based on observation duration.
- Perform field reconnaissance in advance, utilizing whatever mapping resources you may have access to, including:
    - Topographic maps
    - Google Earth Pro®
    - Software-specific aerial imagery
- Pre-plan GNSS sessions. Anticipate the number of sessions and observers. Ensure observation times are adequate.
- Prepare a project mission plan, including equipment checklists and GNSS planning schedules.
- Ensure good communication between the project teams while observations are ongoing.

B.1.5 Post-Processing

Post-Processing (Interactive Using Commercially Available Software):

GNSS static data collected for high precision applications must be post-processed to produce accurate results. Specific workflows for post processing are software/manufacturer dependent; surveyors should consult the user manual for best results.

Post-Processing (Online Positioning):

Today there are several online positioning post-processing programs that can meet certain surveying needs. The requirements for each of these can be slightly different. The following are some of the most common options:

- Online Positioning User Service (OPUS) (National Geodetic Survey). Users should always wait for the publication of the precise ephemeris for the most accurate results when utilizing the OPUS processing network.
- CSRS-PPP (Canadian Geodetic Survey)
- AusPos (Geoscience Australia)
- APPS (NASA/Jet Propulsion Laboratory)

B.2 Establishing Control Networks Using OPUS Projects

B.2.1 NOAA Technical Memorandum NOS NGS 92 (Currently Under Review)

It has come to our attention that the NGS will be releasing a new NOAA document relating to the creation of geodetic control surveys with GNSS, using the recently-released Version 5.1 of OPUS

Projects. OPUS Projects from NGS provides simple management and processing tools for survey projects which involve multiple sites and multiple occupations. This web tool is highly recommended by ASPRS, as it is able to make use of both static post-processing and RTK/GNSS data when establishing survey control networks for least squares adjustment in order to yield coordinates and their estimated uncertainties for ground control points and checkpoints. The link below provides guidance on the classifications, standards, and specifications supporting OPUS Projects 5.1.

https://geodesy.noaa.gov/web/science_edu/webinar_series/standards-specs-opus-projects.shtml

B.3 GNSS RTK Positioning

RTK surveying is a relative positioning practice that measures the three-dimensional vector between two or more GNSS receivers in real-time. One GNSS receiver (referred to as the base station) is set up at a known point with fixed coordinates. The base station transmits its known 3D position and the raw GNSS data it receives to the rover receiver in real-time, and the rover employs both the rover's and base station's GNSS data to compute its position relative to the base station.

RTK surveying requires a consistent and dependable communication link between the two receivers so that the rover receives continuous observation data from the base station. Common communication methods used to transmit data from the base station to the rover are UHF or VHF radio links, cellular network modems, or a combination of these two methods via an RTK bridge.

B.3.1 Base Station Setup

- A clear, unobstructed view of the sky above a 15-degree elevation mask is recommended.
- The base stations should be erected in stable environments.
- All setups should use properly adjusted, leveled, and maintained tripods and tribrachs. Tripod leg weights should be employed where necessary.
- Before sending crews to the field, it is preferable to upload the verified NAD83 geographic coordinates, as well as the ellipsoidal height for the monument to be used for the base station receiver, to the data collector/field controller. This is the most reliable way to set up the base station, as it prevents the field crew from having to choose what datum, map projection, and geoid model to work from, and it circumvents the need to key in coordinates manually. Entering incorrect base station data is the most common error associated with RTK surveying.
- Ensure that the antenna height is properly measured, checked, and verified by independent means. Fixed-height tripods or manufacturer-specific, survey-grade height hooks, which both provide vertical height measurements to the millimeter level, are preferable.

When obtaining data from a real-time reference network, please refer to Section C.2.

B.3.2 Rover Setup

When starting an RTK survey, it is imperative to ensure that the rover is configured to achieve the desired accuracy. The following are important fundamentals that must be confirmed for quality data collection:

- Both the base and rover receivers should be set to track the same satellite constellations (GPS, GLONASS, GALILEO, BEIDOU) and signals (L1, L2, L5 and their equivalents).
- If required, a geoid model may be assigned. Heights observed by the GNNS receivers are ellipsoid heights. Geoid models are used to convert ellipsoidal heights to orthometric heights or elevations. At the present time, the most common geoid model used to convert ellipsoid heights to the NAVD88 datum in the US is Geoid18, although Geoid12B is still used occasionally. It is anticipated that a new vertical datum based on a new 3D Coordinate Reference System (CRS) and a new gravity-based geoid model will soon be adopted by the National Geodetic Survey (NGS).

B.3.3 GNSS Solution Types

- *Autonomous*: When the rover is observing independently without any corrections and is not receiving data from the base. Coordinates gathered this way do not meet survey-grade accuracy standards.
- *Float*: When the data obtained at the rover is not of sufficient quality to calculate a fixed integer position.
- *Fixed Integer*: When the GNSS rover can calculate a fixed integer solution, and the positional results normally fall within the desired accuracy limits. This is the most accurate solution type.

Be mindful that not all survey grade GNSS systems employ the float/fixed method of RTK ambiguity resolution. These alternate methods may determine adequate RTK precisions based on a more rigorous float solution.

B.3.4 Rover Quality Control

- In the field, operators must ensure that all surveyed points meet minimum quality standards as set for the project. Revisiting points to re-survey checkpoints that do not meet minimum quality standards is inefficient; therefore, it is important to ensure appropriate procedures and methodologies are followed before and during data collection. Delivering sub-standard survey data is not at all acceptable.
- Coordinate quality thresholds should be set to meet minimum project accuracy requirements.
- On older equipment, monitor Position Dilution of Precision (PDOP) and Root Mean Square (RMS) values to ensure that quality solutions and measurements are obtained. Most modern GNSS equipment allows the user to set rigorous Coordinate Quality (CQ) standards, and therefore does not require the monitoring of PDOP or RMS values. Depending on the make and model of your equipment, many modern receivers do not display the RMS value, as this value is a computed component of the precision calculations and is handled through a threshold accuracy setting.
- A minimum of three independent measurements with independent initializations should be conducted on each checkpoint. More should be used if necessary. These independent measurements should then be averaged or computed as a weighted mean to arrive at the best

estimate of the checkpoint's true position. During this process, it is crucial that any outliers are eliminated from the solution, as these may require additional independent observations in order to reach a reliable solution.

- It is recommended that a minimum observation period of 180 seconds be collected for each individual observation.

B.3.5 Accuracy Check

- It is always the obligation of the surveyor to use appropriate equipment and procedures to achieve and verify the required accuracy for the survey.
- RTK data collected in the field should always be checked and verified using the manufacturer's proprietary office processing software. Data collected in the field should never be exported directly to an ASCII file without an office QC process to catch any field errors and verify the correctness of the data before export.
- To verify that your base broadcast data and your Coordinate Reference System (CRS) are correct, you should locate and tie in existing NGS data or other monuments with known or published values using the same rigorous observation methodology delineated above. Compare the coordinates published by NGS or other agencies to the surveyed coordinates as derived by your field crews to determine whether they fall within the standards required for the project.

For a more detailed explanation of some of these principles, please refer to Section C.2.

SECTION C: GNSS REAL-TIME NETWORKS (RTN)

The purpose of this section is to set forth best practices and guidelines for using a Real-Time Network (RTN) as the reference stations for ground control using RTK.

C.1 Introduction and Definitions

C.1.1 Network Solutions vs. Single Baseline Solutions

When predicting the coordinate quality values of a control survey, it is important to determine whether the reference being used is a single station (baseline), or a network solution.

The baseline length at which acceptable results can be achieved is sometimes shorter with a single baseline solution than with a network solution. A single baseline solution receives corrections from a single reference station, while a network solution models satellite orbit variations and ionospheric and tropospheric differences/interference using multiple reference stations in the area surrounding the rover, which helps the rover estimate the atmospheric conditions at its location. The distance of a single baseline solution can, however, be extended via cellular networks.

C.1.2 GPS vs. GNSS

When determining what type of accuracy and precision can be expected, it is critical to know not only which GNSS signals and constellations can be tracked by the rover, but also which signals and constellations can be tracked by the reference station/network. It does not matter how many constellations (GPS, GLONASS, GALILEO, BEIDOU) and signals (L1, L2, L5 and their equivalents) the rover is tracking if they are not also being tracked and utilized by the RTN. An RTK solution can only use satellites and signals that are being tracked at both the base and the rover.

C.1.3 VRS vs. MAC(X) vs. iMAX vs. FKP

There are significant differences in the type of RTN and how it affects the calculated rover position:

VRS – Virtual Reference Station: A virtual base station is created close to the rover's position and mitigates baseline dependent errors. The server sends modeled corrections to the rover, but the rover is unaware of the errors for which the VRS is modeling. As a result, there may be some degree of error in the virtual position that the rover is not accounting for, which can result in overly optimistic quality predictions.

MAC(X) – Master Auxiliary Concept/Correction: Correction and modeling data is broadcast from one primary (or master) station and several auxiliary stations to the rover. The modeling is performed by the rover based on its position, the information received from the surrounding stations, the baseline length to the closest physical station, etc. The master station is a physical reference station from which the rover receives corrections that can be traced and repeated.

iMAX – Individualized Master Auxiliary Correction: Based on the MAC(X) concept, but modified for lower bandwidth so that full GNSS, multi-signal messages can be transmitted successfully. Correction data is calculated at the server rather than the rover, but correction info and baseline vectors from the closest physical station are still transmitted to the rover. This allows the rover to predict coordinate quality based on the true baseline length.

FKP – Flachen-Korrektur Parameter: A model of distance-dependent errors is transmitted to the rover, and the calculations are performed at the rover. Because more data is transmitted to the rover than most network correction types, the bandwidth requirements are high. FKP is much more common in Europe and elsewhere than in North America.

C.1.4 Vertical and Horizontal Datums and Broadcast Coordinates

An understanding of reference frames, projections, geoid models, and their various realizations is critical when working with RTNs. The method and frequency of processing and adjusting the physical reference station coordinates can have a significant impact on the accuracy and precision possible at the rover, especially in areas of above-average horizontal or vertical movement.

C.1.5 Vertical and Horizontal Accuracy

The factors and variables at play in estimating and validating vertical and horizontal accuracy in an RTN are numerous, but with an understanding of the network type, rover capabilities, baseline length, best practices, etc. both high accuracy and high precision results can be obtained when using an RTN.

C.1.6 Baseline Length

Weather and ionospheric/tropospheric interference/differences are largely dependent on baseline length, which plays a huge role in the accuracy and precision of all RTK surveying. While network corrections are able to mitigate ionospheric and tropospheric differences to a certain extent via modeling, better coordinate qualities can be expected if the atmosphere through which GNSS signals are being received is similar at both the base and the rover. This is particularly true of the vertical component, due to the difficulty in estimating the tropospheric changes over long distances.

C.1.7 Dilution of Precision, Root Mean Square Error, and Coordinate Quality

The same factors that affect accuracy and precision in traditional base-and-rover RTK surveying affect RTN surveying, but more variables are introduced into the solution for RTN. Technical considerations such as satellite constellations and signals tracked, satellite geometry, and dilution of precision become more complicated when handling baseline lengths in excess of 12 miles.

C.2 Procedures and Best Practices

C.2.1 Coordinate Quality/Root Mean Square Dilution of Precision Guidelines

To keep coordinate quality (CQ) and Root Mean Square (RMS) error values at an acceptable level, factors such as baseline length (especially in network RTK when baselines are not limited by radio range) and Dilution of Precision (DOP) values should be monitored and taken into account. Reliable results are possible at longer baseline lengths when the corrections are coming from a network cluster and modeled for ionospheric and tropospheric differences, as opposed to a single baseline solution in which baseline length has a direct impact on coordinate quality (roughly 1–2 cm plus 1 part per million with most survey-grade GNSS receivers).

C.2.2 Satellite Constellation

Full GNSS tracking of constellations (GPS, GLONASS, GALILEO, BEIDOU) and signals (L1, L2, L5 and their equivalents) on both the network and the rover side will improve results on longer baselines in comparison to GPS or GPS and GLONASS, as most modern GNSS receivers are able to automatically remove noisy or redundant signals from the solution. The more signals there are to choose from, the more lower-grade signals the solution can reject while still maintaining a sufficient number to fix integer ambiguities.

C.2.3 Baseline Length

Depending on the network type and signals being used, longer baselines can still produce reliable results if proper procedures are followed, due to the ionospheric and tropospheric modeling inherent to network RTK corrections. While the network type (VRS, MAC(X), iMAX) will affect the achievable accuracy at the rover, any network correction type will greatly improve rover accuracy when dealing with longer baselines, due to the modeling that is not possible in a single baseline solution.

CAUTION: Be aware that VRS networks can display overly optimistic CQ and RMS values due to the proximity of the rover to the virtual station, as opposed to the true baseline length to a physical station. When using a Virtual Reference Station, the process of reinitializing a fixed ambiguity solution, as described above, may not result in an independent initialization, as recommended. When using other forms of Real-Time Networks, the rover receives corrections from a physical reference station with processed, adjusted, and repeatable coordinates. When using a VRS, the calculated “virtual station” upon which the RTK vector and rover position are based, is not updated until the rover moves far enough away from the virtual station to warrant the calculation of a new virtual station. Biases in the virtual station can be significant, and simply reinitializing the rover will not mitigate or account for biases in the virtual station. Best practice when using a VRS network is to disconnect and reconnect from the VRS as part of the reinitializing process. Upon re-connecting to the VRS, a new “virtual station” will be calculated, allowing for independent measurements to be obtained and averaged.

C.2.4 Occupation Time

Minimum occupation times of 180 seconds are recommended. This gives the rover enough time to improve the solution slightly, and allows some modern units time for secondary measurement engine calculations and checks to be completed, improving confidence in the solution. Please note that seconds are used here rather than epochs, due to the fact that many manufacturers allow epoch rates of less than 1 second (e.g., 20 Hz, in which case a 180-epoch observation would only be 9 seconds long).

C.2.5 Redundant Occupation

A minimum of three (preferably more) occupations should be taken at each checkpoint. It is strongly recommended that the unit be re-initialized in a different location (at least 15 feet different horizontally and at least 2 feet different vertically) between observations to ensure any bad initializations are identified. It is statistically very difficult for two independent initializations, when initialized in different locations, to come up with bad initializations that agree with one another. If, on the other hand, no re-initialization is completed, or if a re-initialization is completed in the same location, the chances of bad solutions that are in agreement increase drastically.

In general, more occupations of a shorter duration (as long as the system is properly re-initialized between each observation) are preferable to fewer observations of a longer duration, due to the accuracy advantages of identifying bad initializations as well as the benefits of point averaging that are possible with multiple observations. However, it should also be recognized that there are two benefits to longer observation times: they may allow the real-time GNSS software to more easily identify and eliminate bad initializations, and they will usually produce slightly better results, as each epoch can be considered a measurement.

C.2.6 Point Averaging

Averaging multiple observations is a critical component of control surveying. Multiple observations from separate RTK initializations and, preferably, under different satellite constellations (e.g., different times of day) allow for the creation of a mean—ideally, a weighted mean based on coordinate quality values. This can not only help identify outliers or bad initializations but may also result in an averaged value closer to the true coordinate than individual measurements are likely to be.

C.2.7 QA/QC

Best practices dictate that all field data be analyzed in the office utilizing QA/QC software—preferably the proprietary software of the hardware manufacturer—to ensure that field data was collected properly. A competent office staff member who understands the QC process and is very familiar with the software being used must independently confirm that measurements were based on initialized or fixed integer solutions, with acceptable CQ and/or RMSE values, individual point averages to include (or exclude) the appropriate measurements per best practices, and other QA/QC routines. It is an extremely dangerous practice to accept and use a file exported directly from a data collector/field controller without any true QA/QC of the raw data and metadata via the appropriate office software.

SECTION D: GNSS REAL-TIME PRECISE POINT POSITIONING (RT-PPP) IN OPEN SKY AREAS

This section explains the fundamentals of and demonstrates best practices for using Real-Time Precise Point Positioning (RT-PPP) methodologies to establish control networks for remote sensing applications.

The following guidelines provide a practical method to obtain consistent, three-dimensional positions using a single rover. This is accomplished with real-time signal augmentation corrections designed to remove system errors due to satellite, atmospheric, and receiver-related influences through an inbound data feed.

D.1 Real-Time Precise Point Positioning (RT-PPP)

Real-Time Precise Point Positioning (PPP or RT-PPP) is an alternative to base-and-rover RTK, RTN-based Post Processed Kinematic (PPK), and static surveys, and it utilizes a hybridization of these methodologies. PPP relies on access to precise satellite orbit and clock products received through a data stream either from the satellites themselves or through an internet-based subscription. This data stream removes the need for a base station or a two-way connection to a real-time network to alleviate the broadcast and system positioning errors associated with a single roving receiver configuration. One benefit to utilizing a standalone receiver is the removal of the required tied baseline, thus resulting in a coordinate based on the satellite geometry instead of a conventional coordinate derivation relative to a base station as calculated in an RTN solution or similar RTK system.

D.1.1 PPP Convergence

Convergence relates to keeping positioning errors within a tolerable level, in order to create a final coordinate solution of acceptable quality. This will vary depending on the required accuracy of the control point.

The time the receiver takes to converge is known as the convergence time. It is usually between 1 and 20 minutes.

D.1.2 Rover Setup

- A clear, unobstructed view of the sky above a 15-degree elevation mask is preferred.
- Efforts should be taken to minimize the introduction of signal blockage and multipath errors generated from surfaces reflecting signal to the receiver (e.g., trees, buildings, etc.).
- All setups should be performed with adequately adjusted, leveled, and stable tripods and tribrachs. Due to potentially long convergence and observation times (up to 15 minutes or more) a rod and bipod configuration may not provide sufficient stability and is not recommended.

- Ensure that the proper datum and projection are pre-selected, and that the antenna height is appropriately measured.
- PPP coordinates are computed from the current epoch in a globally-based reference frame, such as the International Terrestrial Reference Frame (ITRF), and then transformed to a fixed epoch within the selected coordinate system of the user. This transformation may introduce errors to the final coordinate solution due to inaccurate correlations between some coordinate systems. Changes to the desired datum and projection are not easily made after the fact. If a specific project-related coordinate system and datum are not specified, follow the National Spatial Reference System (NSRS) recommendation for the current coordinate system guidelines.

D.1.3 Rover Quality Control

- Observations can begin once convergence has been achieved.
- Coordinate auto-store accuracy thresholds should be lower than project accuracy requirements.
- Routinely re-measure previously measured points to ensure quality.
- To ensure quality, a minimum of three independent measurements should be conducted for each point. Each measurement should have a different initialization, and each one should meet the required project accuracy specifications.
- It is recommended that a minimum observation time of 300 seconds or more should be collected for each observation. Because of how RT-PPP solutions are derived, observation times will need to be longer than the ones associated with RTK or RTN solutions in order to achieve acceptable results.

D.1.4 Accuracy Check

The surveyor must always use appropriate equipment and procedures in order to achieve the required accuracy for the survey.

To ensure that appropriate results are being achieved, locate and observe existing NGS or other known geodetic monuments using the same rigorous observation methodology delineated above. Compare the coordinates published by NGS or other agencies to the surveyed coordinates as derived by your field crews to determine if they match the standards required for the project.

D.1.5 PPP Limitations

- Convergence times may vary greatly. They typically range from 1–20 minutes, depending on the performance of the correction services within the rover’s region.
- The achievable accuracy for PPP may be variable, but generally falls in the range of 1–2 cm horizontal and 3–5 cm vertical under optimal conditions if multiple (at least three) observations are made and rigorous quality control methodologies are followed.
- Current PPP broadcast correction services may only include certain constellations unless certain access fees are paid. The Real-Time Service (RTS), provided by the International GNSS Service (IGS), is offered as a GPS-only operational service.

SECTION E: CONVENTIONAL SURVEYING FOR VVA CHECKPOINTS UNDER TREE CANOPIES

The purpose of this section is to lay out the best practices and guidelines for utilizing a total station to measure Vegetated Vertical Accuracy (VVA) assessment points under tree canopies. This section is only meant to highlight these guidelines and is not a replacement for adequate education and experience in surveying practices.

A total station is a modern surveying instrument that measures horizontal and vertical angles, along with slope distances between the total station and an object (usually a prism on a range pole). Total stations have onboard microprocessors to assist with level compensation and allow for the station to calculate averages for multiple angle and distance measurements. Total stations are rated based upon their angular accuracy and distance measuring capabilities. Common total station angular accuracies include 1”, 3”, and 5”, with 1” (“ here is arc second) being the most accurate.

A total station may be combined with a data collector to allow for real-time conversion of the angular and distance measurements into X, Y, Z coordinates in the form of eastings, northings, and elevations. Some total stations may have onboard data collection software that can be accessed through a user interface, and more advanced total stations have robotic tracking capabilities (for potential single-person operation) and some limited scanning capabilities. It is important to note that total stations are precision instruments that require regular adjustment and calibration.

This section assumes that control points will be established for a total station with GNSS RTK or RTN methods. Please see the sections above for more information on real-time GNSS surveying techniques.

E.1 Temporary Control Points for Total Stations

A temporary control point is a semipermanent point with a known northing, easting, and elevation that can used as a total station occupation point, backsight point, or checkpoint. This section assumes that temporary control points will be measured in real time with GNSS methods. Please see the sections on real-time GNSS for more detailed information on these technologies.

Temporary control points established using real-time GNSS techniques should be established in locations with clear views of the sky and limited multipath issues. The control points should be placed in relatively stable materials to ensure that the position of the control point does not change over the duration of the survey. A minimum of three temporary control points should be established for total station work, consisting of an occupation point for the total station, a backsight point, and a checkpoint to verify that horizontal and vertical errors are within acceptable limits. The occupation point should provide a good view into the area of interest where the VVA points are to be collected. The backsight point should be located as far away from the occupation point as is practical to minimize the angular errors in the data collection.

E.2 Total Station Data Collection Recommendations

The following is a list of recommended best practices for collecting data with a total station:

- Verify total station calibration.

- Follow a systematic data collection methodology to ensure adequate and accurate data is collected.
- Verify the height of the range pole and, if necessary, adjust before beginning the survey.
- Ensure that the range pole is plumb (in vertical adjustment).
- Use a bipod or tripod to set up the total station.
- Direct and reverse measurements should be taken and averaged when backsighting and collecting points (foresights).
- Utilize checkpoints.
- A hardened point should be used on bottom of the range pole for temporary control points and a “topo foot” should be used to collect VVA or other topographic points.
- If the height of the range pole is changed during data collection, ensure that the data collector or onboard software is updated to reflect the correct height.
- No data collection points should be farther away from the occupation point than the distance between the occupation point and the backsight point. This is especially important when establishing GCPs and NVA checkpoints.

E.3 Traversing into Vegetated Areas

When it is necessary to traverse into vegetated areas, the guidelines above still apply. As the total station is a line-of-sight instrument, planning and forethought is required to set subsequent instrument occupation points. In addition to finding an occupation point that allows adequate visibility of the region of interest, the previous occupation point needs to be visible. It is also recommended that the checkpoint remains visible, or another checkpoint is set. When additional traversing is required, a traverse closure and adjustment routine should be used.

SECTION F: TERRESTRIAL SCANNING OR MOBILE MAPPING FOR GCPS AND NVA CHECKPOINTS

This section is intended to provide best practices and guidelines when utilizing existing point cloud data collected by either mobile mapping systems (MMS) or static terrestrial-based lidar for the purposes of controlling or verifying aerial lidar and photogrammetrically-derived products. The cost to acquire MMS or terrestrial-based lidar data for the sole purpose of establishing control points/checkpoints would be prohibitive; therefore, this section will focus on the re-use of existing ground-based point clouds to extract ground control points and/or checkpoints.

An MMS is a vehicle-mounted array of sensors and a computer that can incorporate three-dimensional positioning with data collected from a variety of active and passive sensors. The most common sensors in MMS arrays are inertial navigation systems (INS), which consist of integrated GNSS receivers and inertial measurement units, lidar, and cameras. As the vehicle moves, a constant stream of three-dimensional coordinates and sensor data are sent to the computer. This allows the computer to relate the position of the vehicle to the measurements taken from the sensors, and then compile one large point cloud that contains all of the measurements acquired.

Static terrestrial lidar is mounted on a tripod and positioned at various locations. The scans collected at each location are combined into one point cloud by registering the scans together. This is accomplished either by aligning common features that appear in multiple scans, or through registration of known coordinates based on the features visible in the individual scans.

F.1 Permitted Use

Determine if the point cloud was collected for another client, or if deliverables produced from that point cloud were contracted by another client. It is a good idea to seek permission from the client for whom the data was originally collected before use.

F.2 Selecting a Suitable Point Cloud

There are many factors that can affect how suitable previously-collected data may be for reuse in subsequent projects. Care should be taken when selecting a suitable point cloud to use. Below are some areas that should be investigated to determine if the point cloud is suitable:

- What sensor was used for the ground-based collection? Do the ground-based sensor specifications meet/exceed the airborne project's specifications?
- Determine if the point cloud density meets/exceeds the current project specifications.
- Are photo-identifiable points easily recognizable?
- Is the point cloud density high enough that features are not hidden due to the point spacing?
- Was the point cloud constrained to survey control, or was it uncontrolled? If the former, what survey techniques were used to establish the control?
    - Static GNSS, RTK or RTN GNSS, leveling, total station?
    - Were redundant measurements obtained?
- Review the point cloud registration statistics. These statistics should exceed the requirements for the airborne project.

F.3 Spatial Reference System

The CRS of the existing dataset will need to align with the aerial product's CRS, or else a transformation will need to be applied. To ensure the data aligns between the two projects, the horizontal datum, projection, adjustment date, epoch, vertical datum, and geoid model should be identified. If one of the products has been taken by ground coordinates, the combined factor and scaling origin also need to be determined in order to reverse the process.

F.4 Verifying the Point Cloud

Perform a site visit to see if any noticeable alterations have taken place since the point cloud collection:

- Have any of the roads been realigned?

- Does any of the pavement look different or newer?
- Is there evidence of earthwork or construction?
- Has any pavement restriping occurred?

F.5. Field Checking the Point Cloud

It is prudent to check ground-based lidar by surveying features in the point cloud. This can bring to light any discrepancies in the stated horizontal and vertical datum, as well as potential alterations to the site. Compare the derived position of the point cloud to the recently-surveyed position and determine if the error is within the project specifications.

F.6 Extracting Control from the Point Cloud

Once it has been verified that the point cloud represents the current conditions and is in the same Coordinate Reference System (CRS) as the aerial project, the extraction process can begin.

When selecting points from a point cloud that was collected on the ground, be sure that the points selected have an unobstructed view of the sky. Think about tree canopy, building overhangs, and shadows that would prevent the point from being recognized in the aerial data.

For aerial products, it is a good idea to locate features that do not have a sudden elevation change nearby. Points at the edge of a roadway that has an adjacent ditch, at the top or bottom of a wall, or at a headwall are poor choices. Depending on the aerial lidar point spacing or shadows in the photography, these points may not be discernable in the aerial data.

- It is a good idea to make sure the point cloud has been cleaned of points that were measurements on transient objects, vehicles, or pedestrians present during the collection.
- When selecting points to extract, rotate the perspective to verify the location selected is the intended location. Often, different parts of the point cloud are selected unknowingly, but rotating the view will reveal if the correct position was selected.

CAUTION: While Terrestrial Scanning and/or MMS technologies may be useful when establishing ground control points and checkpoints, they must be employed carefully and only when all the above-noted checks and balances are in place. Indiscriminate use without some degree of ground verification by the surveyor can lead to disastrous consequences.

ADDENDUM III: BEST PRACTICES AND GUIDELINES ON MAPPING WITH PHOTOGRAMMETRY

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FIGURES

TABLES

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PURPOSE

The objective of this Addendum is to provide general best practices and guidelines for photogrammetric workflows, focusing on positional accuracy, frame cameras, and fixed-wing platforms. These guidelines are intended to be sensor and manufacturer agnostic.

This Addendum establishes a common framework for the production of photogrammetrically-derived data and the use of aerial photography for surveying and mapping, in order to establish a resource for producers and users of photogrammetric data that complements other guidelines and standards published by ASPRS, such as the ASPRS Positional Accuracy Standards (Edition 2, Version 2.0), which shall be referred to as the ASPRS Accuracy Standards in the rest of the Addendum.

These best practices and guidelines are intended to:

- Provide a common nomenclature for the components of photogrammetric workflows
- Describe the various inputs and outputs in photogrammetry
- Provide a framework for the standard photogrammetric workflow in logical steps
- Provide examples of topics
- Suggest best practices to enable quality project designs

SECTION A: CAMERA SYSTEMS

Aerial camera systems are precision image-acquisition systems. Cameras can be classified as metric or non-metric.

Metric cameras have stable interior elements designed for precision imagery acquisition. The interior elements include: the focal length of the camera, pixel size, principal point, and camera lens distortion parameters. Metric aerial cameras are generally placed in a camera mount attached to a shock-absorbent airframe. The mount may also be equipped with a gimbal system, to allow the camera to accommodate the aircraft roll and pitch and to compensate for the heading direction. Some metric cameras have forward image motion compensation to capture sharper images.

Non-metric cameras are usually used on unmanned aerial systems (UAS). They are off-the-shelf cameras designed for general-use photography. Interior elements may be unknown or only partially determined. The construction is less stable and more influenced by external atmospheric conditions including temperature, pressure, and humidity. It is highly recommended to use a global shutter in these types of cameras to avoid artifacts due to motion.

The aircraft should have onboard dual frequency Global Navigation Satellite System (GNSS) (L1 and L2) capabilities and use Post-Processed Kinematic (PPK) or Real-Time Kinematic (RTK) for camera positioning. In addition, the GNSS needs to be tied to a camera system via a well-defined electronic mid-exposure pulse (EMP) so that the three-dimensional coordinates of the exposure station can be accurately measured the moment the shutter is opened.

Aerial cameras are often categorized according to image frame size format (large, medium, or small). Traditional precision photogrammetry typically uses a large format camera. Some large format digital aerial camera systems feature multiple cameras with CCD or CMOS sensors designed to efficiently and accurately cover large sites while accounting for the necessary acquisition altitude. Dual sensor systems that incorporate airborne lidar may also feature a medium format camera mounted to the platform.

SECTION B: CAMERA CALIBRATION

Camera calibration is essential for obtaining accurate, verifiable positioning measurements suitable for photogrammetric mapping. The primary calibration method is geometric calibration, which can determine and validate camera elements including the focal length, principal point, and camera lens distortion, which are called interior orientation parameters. These interior orientation parameters can be used to model the image space. Factory calibration and in situ calibration are the two most common geometric calibration types.

B.1 Factory Calibration

Factory calibration is conducted by the manufacturer in a laboratory. The sensor calibration may consist of several other calibration methods, in addition to geometric calibration. A digital camera calibration report provided by the aerial camera manufacturer typically includes geometric, radiometric, shutter, and electronic and sensor calibration parameters, with corresponding accuracies. The calibration report also typically contains details about the methods used to derive the orientation elements.

Factory geometric calibration entails capturing redundant imagery across multiple targets in a controlled, enclosed environment at specific ambient conditions. The collected imagery and target measurements provide necessary bundle adjustment data to solve for the camera’s interior orientation parameters. The ability of the cameras to effectively capture and record light is also important to acquiring quality imagery, so radiometric calibration is often performed during the laboratory-based validation. Radiometric calibration measures the sensor’s response and output relative to the light source and involves taking multiple images of targets with precision light sources.

The aerial data producer may need to take the camera in periodically to be re-calibrated in a laboratory setting, especially if the camera is not producing satisfactory results or has seen frequent use. Data producers should follow manufacturer recommendations on how frequently this type of calibration should be repeated.

B.2 In Situ Calibration

In situ calibration is another method of aerial sensor calibration, in which data is collected during an airborne mission under field site conditions using a ground control network. The resulting data is processed with supporting software, then used to model the aerial camera orientation parameters as they existed during data acquisition.

The ASPRS Guidelines for the Aerial Camera In Situ Geometric Calibration of the Aerial Camera System, published in October 2013, highlight that aerial cameras are part of a remote sensing system in which additional components may contribute to the total potential error, and thus must be considered during the aerial system calibration. Airborne systems often include a GNSS antenna and an Inertial Measurement Unit (IMU), whose centers do not coincide. In addition to these lever arms, the angular relationship between the camera coordinate system and IMU body frame, called camera boresight also has to be calibrated with high accuracy. In situ calibration can address both the interior orientation and camera boresight angles.

Non-metric cameras, including those in UAS, lend themselves to in situ calibration, or self-calibration, as the component mounting is less stable and orientation elements may vary depending on the flight conditions. Software that accommodates self-calibration in the bundle adjustment must be used. Alternatively, repeated lab calibrations may be necessary to determine orientation elements.

SECTION C: FLIGHT PLANNING CONSIDERATIONS

Successful execution of a photogrammetry project requires careful flight planning that considers the project requirements and accuracy specifications. The first decision to be made is to select an imagery sensor that is cost effective and appropriate to the needs of the project, including the project specifications, deadlines, and required deliverables.

Site conditions such as area, vegetation density, and topography must be considered when selecting a sensor, especially in respect to project accuracy requirements and deliverables. If, for instance, the site contains heavy vegetation where some light can filter through the cover, a dual camera and lidar system may be more appropriate than a photogrammetry sensor. If the site is a large, feature rich area that needs to be mapped at higher horizontal accuracies while accommodating for factors such as sun-angle and lighting conditions, a photogrammetry sensor may be more suitable. If the site is smaller and requires higher-resolution imagery, a UAS may be more appropriate if the site meets the necessary safety standards.

An available digital elevation model (DEM) of the project area and boundaries should be used for flight planning, in tandem with the appropriate flight planning software. The flying altitude should be determined according to the project’s required ground sample distance (GSD), accuracy specifications, and expected deliverables. In the event lidar data is collected together with photogrammetric imagery, then lidar point density should be considered when planning the flying altitude. Some of the other major flight planning considerations include the following:

C.1 Flight Line Layout

Flight line layout should be determined based on flight plan efficiency, and it should also take into account factors such as airspace restrictions. Production lines should usually be flown in alternating directions, as this is often more efficient and may aid in post-processing. Any planned cross lines should be included in the flight plan. Flight line layout should also consider IMU-related maximum time on-line requirements, based on the manufacturer's recommendations.

To ensure proper coverage, each flight line should extend over the project boundaries according to the project specifications.

C.2 Endlap/Forward Lap

Endlap (or forward lap) must be between 55% and 85% coverage frame-to-frame, depending on project requirements. Increasing the endlap increases the number of image point looks, which will increase the aerial triangulation (AT) accuracy. If Structure from Motion (SfM) is used, it is recommended to use a higher endlap (75% to 85%). Endlap can be adversely affected by the flight trajectory of the aircraft due to crab or drift caused by wind. The endlap should be increased in steep terrain to avoid gaps between images.

C.3 Sidelap

Sidelap should preferably be 30% or more, depending on project requirements, and it should be increased for areas with significant elevation variation. If SfM is used, it is recommended to use a higher sidelap (65% to 75%). Planned endlap and sidelap should also take into account whether the camera system uses a gimbal mount.

C.4 Flight Tolerances

Acquisition tolerances will vary depending on the flight plan and the sensor used. The flight plan should include tolerances for the following:

- Maximum horizontal deviation from planned flight line
- Maximum vertical deviation from planned flight line altitude
- Maximum crab while on-line
- Maximum bank angle during turns—this affects GNSS satellite loss of lock
- Maximum time on-line based on the manufacturer's recommendations

C.5 GPS Base Station Coverage

There are various GNSS/IMU processing techniques in use for airborne GNSS/IMU processing. Some use one or multiple base stations for differential GNSS processing, while others require a subscription service to provide correction parameters. Depending on the required accuracy, Precise Point Positioning (PPP) may also be an option.

The method of airborne GNSS/IMU processing should be based on project requirements, accuracy specifications, and project location. If the processing is planned using NGS Continuously Operating Reference Stations (CORSs), then proper coverage should be checked ahead of time, taking into account any maximum baseline requirements. All CORSs should be checked ahead of time to ensure that they are operational. If CORS coverage is not sufficient in the project area, then it may be necessary to set up base stations.

C.6 Environmental Factors

Environmental factors may include: vegetation foliage requirements (leaf-off versus leaf-on collection); minimum sun angle requirements (typically 30-degree sun angle at minimum); and terrain conditions such as flooding, snow cover, tidal conditions, etc. Collection should be avoided during hazy conditions, cloud cover, and smoke.

C.7 Ground Control Points and Checkpoints

A plan for ground control points (GCPs) and checkpoints should be established during flight planning. This includes determining the number of points, the approximate locations of GCPs and checkpoints, and the required accuracy of the points based on project specifications. More details can be found in the Ground Control section of this Addendum.

In addition to the flight planning considerations described above, the flight plan should also consider airspace restrictions, as well as any necessary permits that may need to be obtained.

SECTION D: DATA ACQUISITION CONSIDERATIONS

When acquiring remote sensing data, including photogrammetric data, it is important to keep the following factors in mind:

D.1 Environmental Factors

Weather affects both operational safety and acquired image quality. Photogrammetric data is best collected during clear weather conditions during optimal sun angles, as imagery that is free of shadow and haze is the best basis for producing accurate photogrammetric models and deliverables.

Other noteworthy environmental factors include vegetation foliage requirements, minimum sun angle requirements, and terrain conditions. Collection should be avoided during hazy conditions, cloud cover, and smoke.

D.2 Checks to Perform Before Flight

The data producer should use a pre-flight checklist. There are a few basic checks that should be performed before flight. Sensor manufacturers typically provide a checklist to follow to make sure the sensor is operational and ready for data acquisition, such as: ensuring the sensor cover was removed, the glass was cleaned, and that there is enough space on the storage media.

If a Continuously Operating Reference Network (such as CORS) will be used for GNSS/IMU processing, then the crew should verify that the relevant CORSs are operational.

It is recommended that the data producer has a pre-flight checklist to follow.

D.3 Flight Log

A flight log should be filled out for each mission. Flight logs should contain information such as the date of collection, project name, mission number, the names of the pilot and operator, sensor used (including its serial number), aircraft tail number, list of collected flight lines, any comments about problems encountered during flight, etc.

D.4 Collection Approach

The Position and Orientation System (POS) manufacturer's recommendations should be followed for specific GNSS and IMU initialization procedures. For IMU initialization, an S-curve pattern should be flown before the first line and at the end of the mission, as directed by the POS manufacturer. If CORSs are used, it is best practice to fly over the CORS for initialization.

Production lines should generally be flown in alternating directions, as this is often more efficient and may aid in the post-processing.

During data acquisition, the navigation tolerances used for flight planning should be taken into consideration; if navigation tolerances are exceeded, the collected data may fail to meet project requirements.

Areas that have access issues or a limited acquisition window due to environmental factors should be prioritized.

D.5 Ground Control Survey Considerations

Depending on the project accuracy requirements, it may be necessary to set and survey ground control. The site will need to be accessible for setting and surveying ground control. Positive GNSS reception is needed to acquire accurate data, but this could be negatively affected by factors such as mountainous terrain. The field team can help confirm the GNSS reception before mobilization and data collection.

SECTION E: INITIAL QUALITY CONTROL OF COLLECTED DATA

Prior to additional processing, an initial evaluation should be performed on the dataset to identify any issues that may need to be corrected. This initial evaluation can be performed in either the field or the office.

The collected imagery should be inspected for clouds, shadows, or other environmental factors, as well as exposure problems, turbulence, any potential sensor anomalies or download issues, or (in the case of vertical image collection) excessive crab or tilt.

The real-time or processed airborne GNSS/IMU can be used to create image footprints on the ground, or the images can be orthorectified using an available digital elevation model (DEM) to generate orthophotos. The image footprints or orthophotos can then be overlaid on the project boundary and flight plan to check for proper coverage and achieved sidelap/endlap, as well as any problems such as gaps between flight lines or missing frames. Depending on the findings, reflights may be necessary.

Affected flight lines or flight line segments should be reflown if they do not meet the project requirements. It is good practice to establish a methodology to keep track of accepted/rejected flight lines and reflights.

SECTION F: GROUND CONTROL POINTS

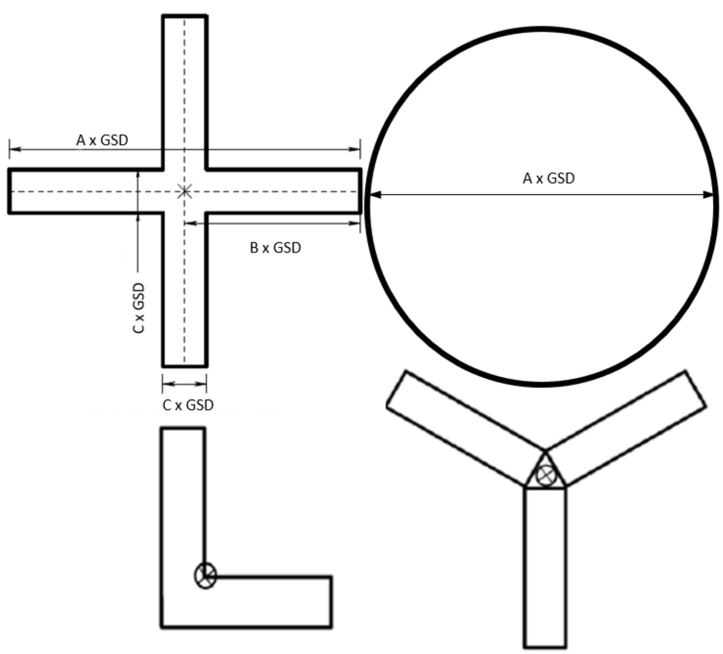
F.1 Accuracy Requirements for Ground Control and Checkpoints

As stated in Section 7.9 of these Standards, ground control points used for aerial triangulation and checkpoints used to check the achieved accuracy should have higher accuracy than the expected accuracy of the derived products.

Surveyors should refer to Addendum II: Best Practices and Guidelines for Field Surveying of Ground Control and Checkpoints for more information.

F.2 Shape and Size

Figure III.F.1 shows a typical X-type photo target, including the recommended dimensions. Examples of circular-type, L-type, and Y-type targets are also provided.



	Open Area	Wooded Area
A	20	24
B	10	12
C	2.5	3

Figure III.F.1 Target Design and Size

F.3 Placement

Set the control on a hard, level surface. The target should be painted with flat black and white paint. If this is not possible, a cloth target should be used instead. Avoid placing the control near tall objects like trees or buildings, as these may obstruct the image of the target in one or more exposures, and can also cause multipath effects if GNSS techniques are used to establish coordinates. Avoid shaded areas during the time of photography.

When available, photo-identifiable (PID) points can also be used as ground control points and checkpoints. This requires the PID points to be identified both on the ground and in the photographs. PID locations must satisfy several conditions to ensure an acceptable accuracy level:

- They must be sharp, well-defined, and positively identifiable in all photos.
- They must lie in level, unobstructed locations.
- They must have photos and thorough, accurate, detailed written descriptions.

Suitable PIDs include but are not limited to: the intersections of parking lot markings, sidewalk corners or the intersections of sidewalk edges; the center of maintenance hole or valve covers; and certain road paint stripes. Using intersection of paint markings and centerpoints is preferred over edges/corners, because those can be identified and measured with higher accuracy on the imagery. The required GSD of the image must be considered when deciding on what types of PIDs may be suitable for the project. A PID type that is suitable for one GSD project may not be appropriate for a larger GSD project.

F.4 Distribution of Ground Control Points

The distribution of ground control points using GNSS photogrammetry depends on the geometry of the project (strip or block) and how the GNSS data is collected. All projects should have at least 30 checkpoints, as per the ASPRS Accuracy Standards.

Linear or strip projects – Since the GNSS data controls the strip's center, wing control points are required every 6 to 8 models (see Figure III.F.2).

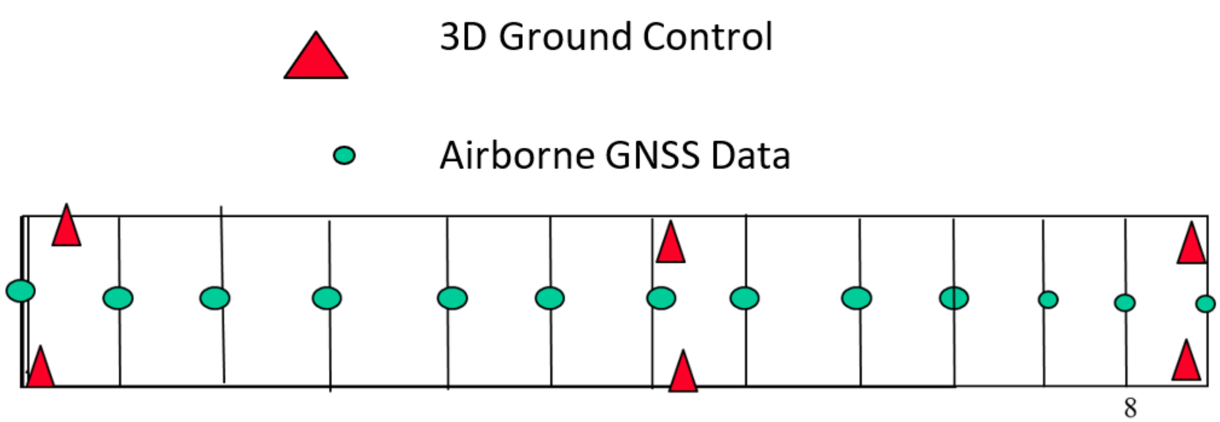


Figure III.F.2 Ground Control for Strip

Area-wide or block projects – Ground control requirements depend on how the GNSS data is collected during image acquisition:

1. Project with continuous GNSS data: The GNSS data is collected without losing lock (Figure III.F.3). This requires flat flight turns. Due to the short flight duration, projects flown with

unmanned aerial systems (UAS) may have this property. The AT software will need a block drift parameter or 3D transformation option during the data processing.

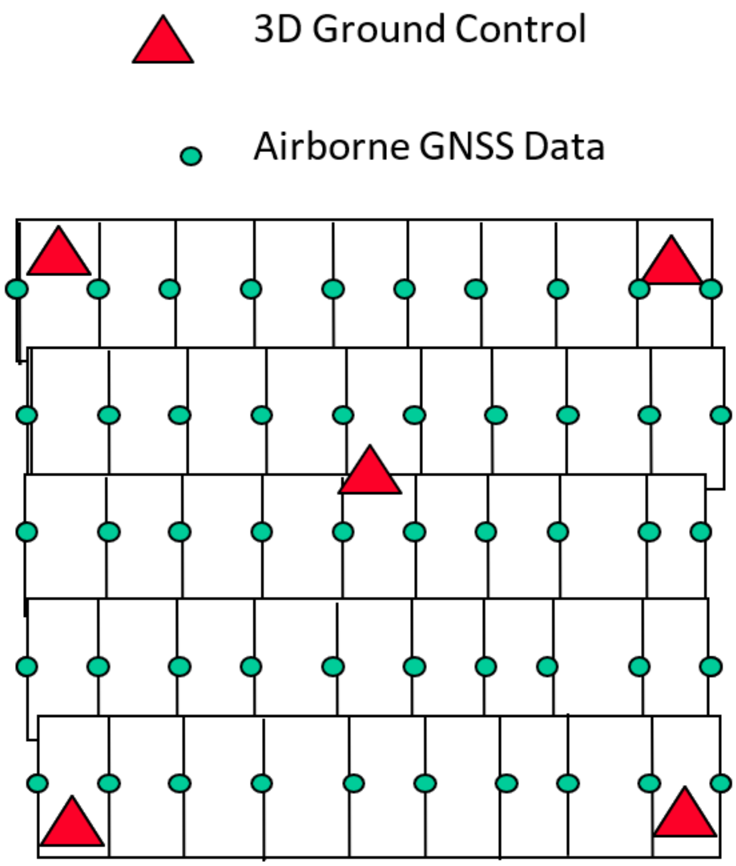


Figure III.F.3 Ground Control for Block Without Losing GNSS Lock

2. Project with a possible loss of lock for GNSS data: Sharp turns of the airplane will cause each strip to have an independent 3D transformation system or drift parameters. This will require the AT software to have a strip drift option. In this case, it is recommended to add a cross-flight line perpendicular to the project's flight lines at each end of the project boundaries (Figure III.F.4).

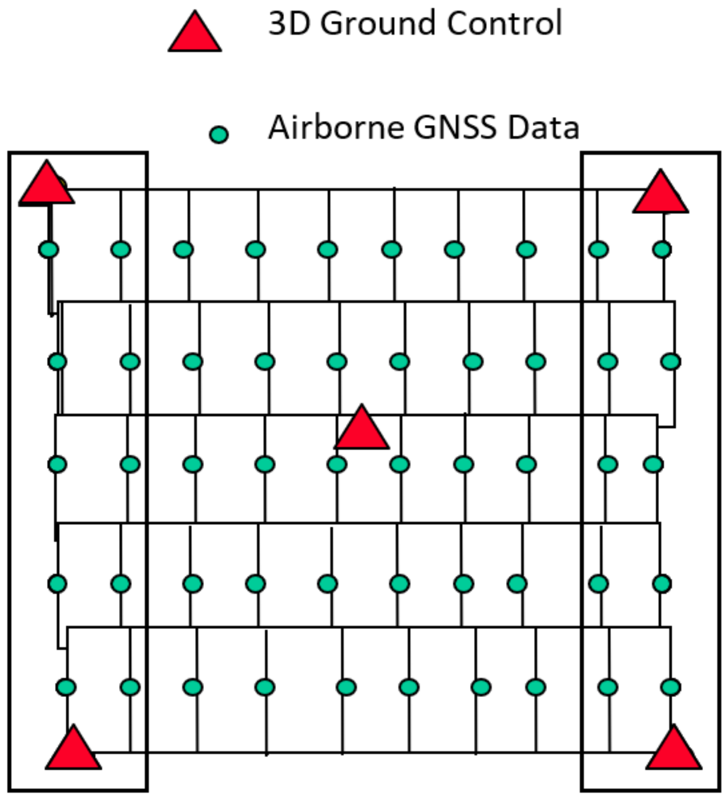


Figure III.F.4 Ground Control for Block with Possibility of Losing GNSS Lock

Sample wide area projects: Three sample projects with various geometric constraints are presented below:

1. Project A: Conventional AT block (Figure III.F.5). 3D control points need to cover only the block perimeter and only height control points are needed in the block interior, but due to current survey techniques the interior GCPs are normally also surveyed as 3D GCPs. When GNSS AT is applied (Figure III.F.6), a significant reduction in the amount of ground control is observed. Any control points added in the block interior can serve as checkpoints.

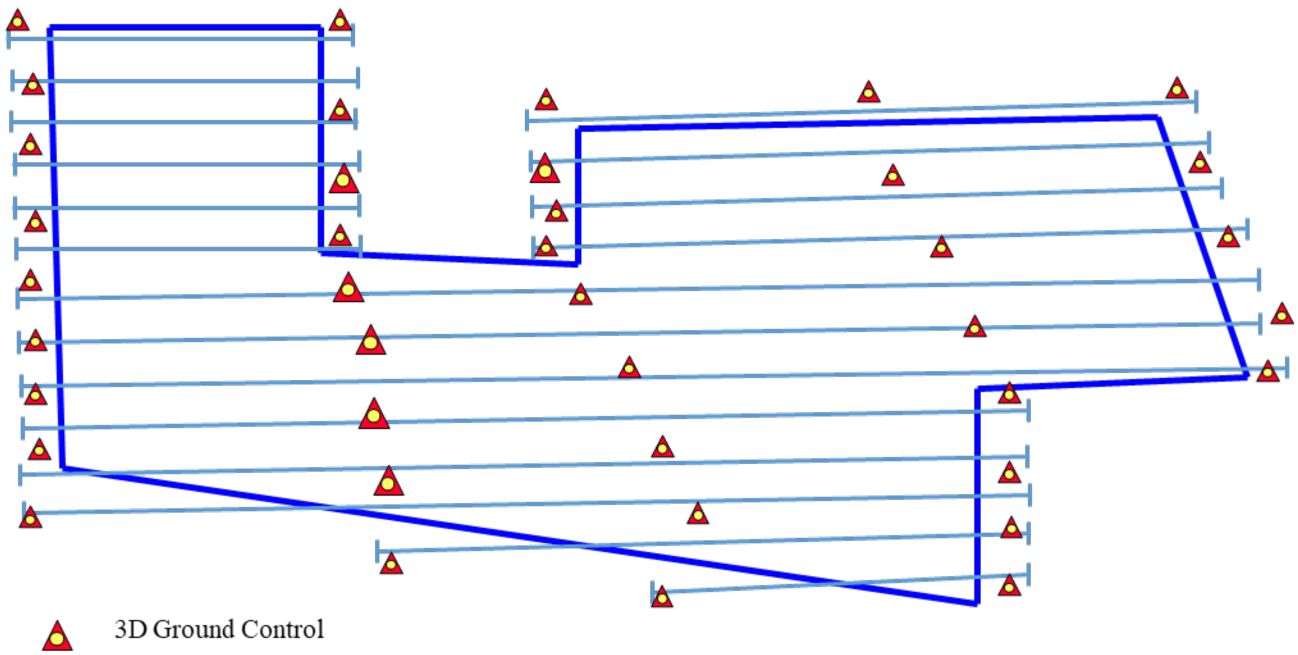


Figure III.F.5 Conventional AT Block

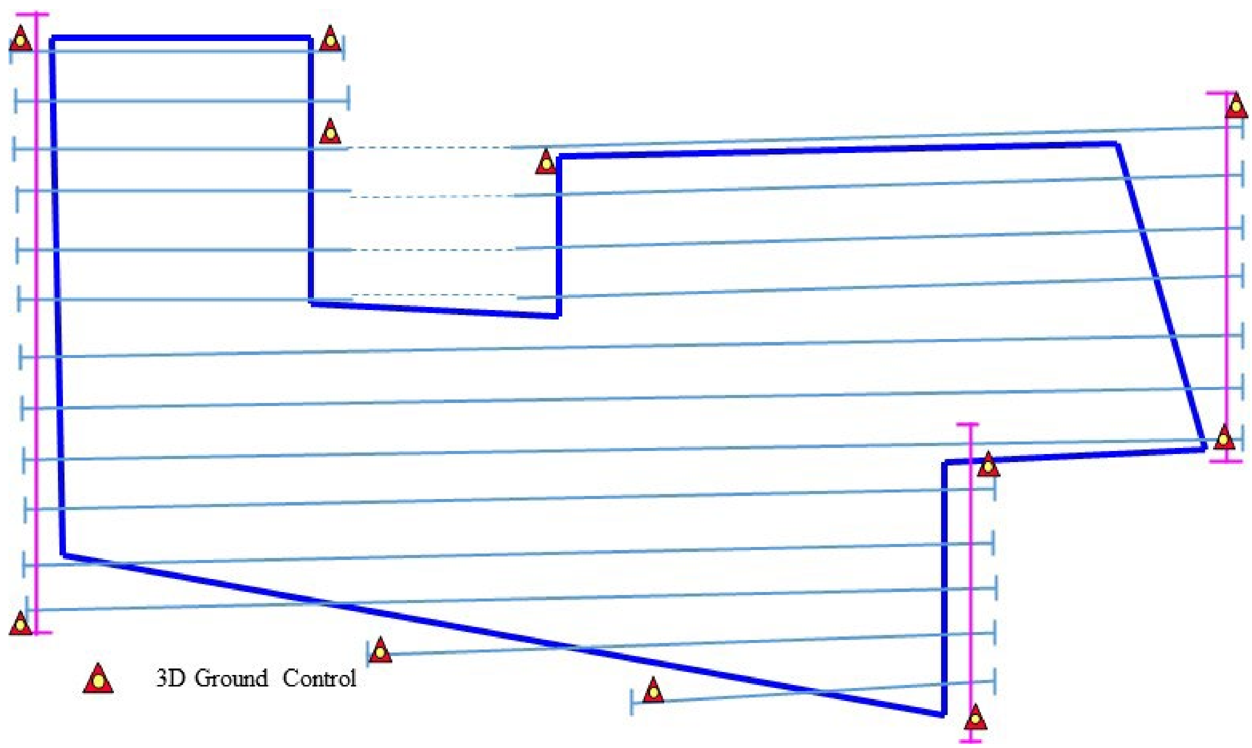


Figure III.F.6 GNSS AT Block

2. Project B: This project (Figure III.F.7) effectively uses cross-flights to accommodate GNSS drift. Checkpoints have been provided in the block interior.

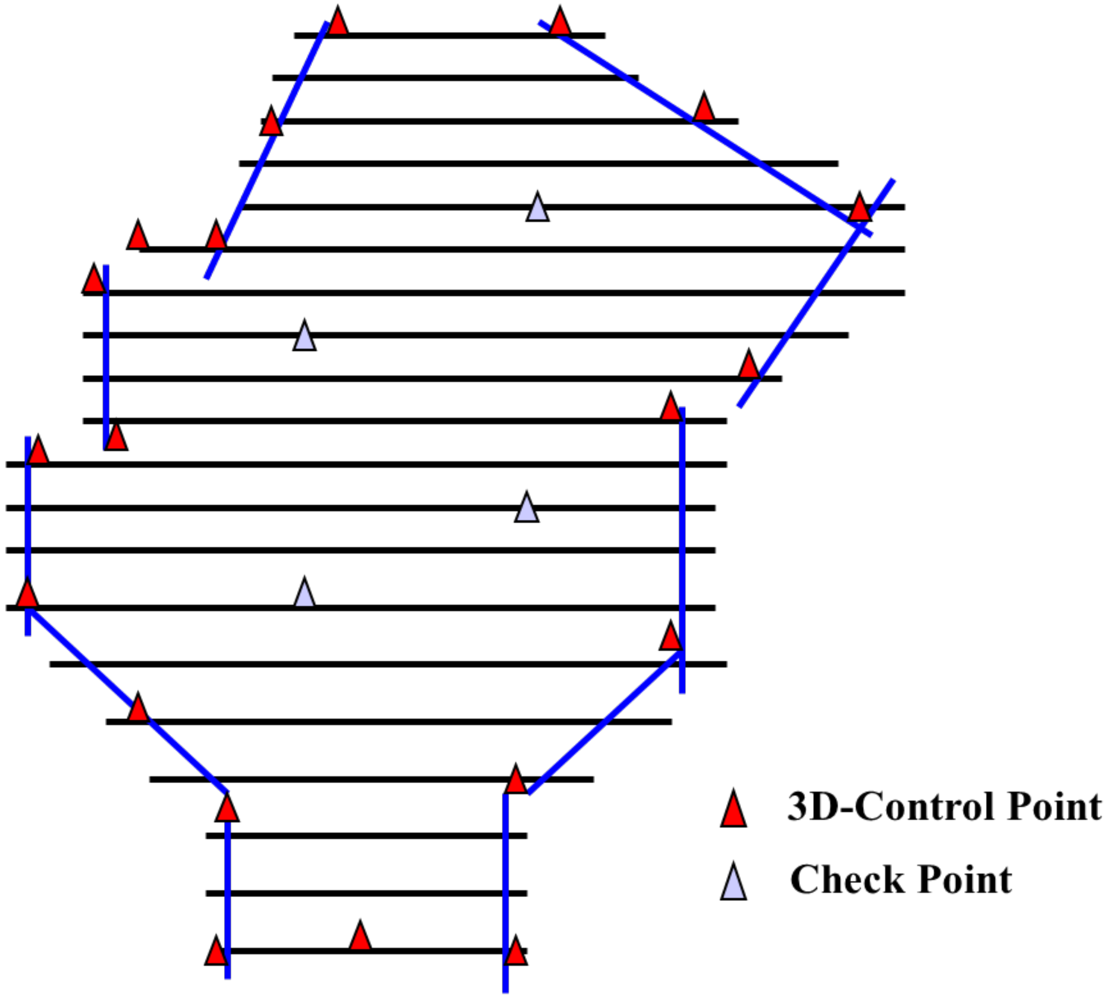


Figure III.F.7 AT Block with Cross-Flights for GNSS Drift

3. Project C: This project (Figure III.F.8) shows the effect of water areas on the flight plan and ground control points.

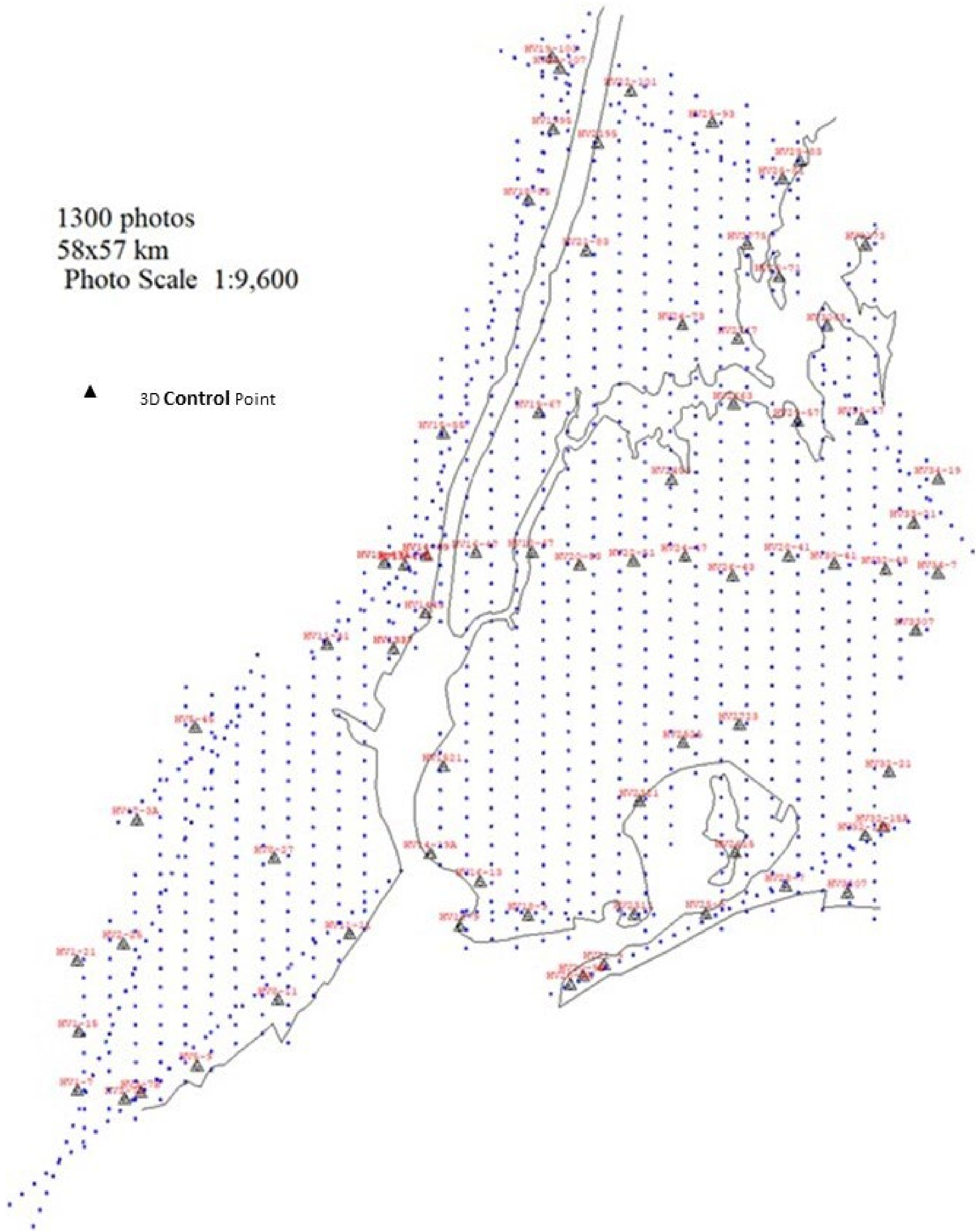


Figure III.F.8 Project with Water Areas

SECTION G: AERIAL TRIANGULATION FOR FRAME CAMERAS

The objective of aerial triangulation (AT) is to densify a sparse network of ground control points to accurately compute the exterior orientation parameters of overlapping images (usually vertical). The exterior orientation parameters are computed for use in three-dimensional mapping, orthophoto generation, and Digital Terrain Modeling (DTM).

Aerial triangulation may be achieved by applying simultaneous bundle block adjustments to the project control points (including ground and image coordinates), and tie points. Tie points are corresponding points on overlapping images, and tie point coordinates are generated either by correlation, or by using SfM computer vision techniques. To reduce the number of ground control points, airborne GNSS-derived camera position and IMU-derived camera orientation parameters can be incorporated into the bundle adjustment.

Alternatively, the combination of GNSS and IMU can provide the image exterior orientation without the need for aerial triangulation. This is called direct georeferencing or direct positioning, and no ground control is needed. The IMU is required only for the push broom camera type. While direct georeferencing is attractive, it is not considered best practice when highly accurate product generation is required.

SECTION H: AERIAL TRIANGULATION PROCESSING

There are many aerial triangulation software packages on the market, and each software package has unique adjustable parameters and user input requirements, including the format of the camera calibration parameters, and how to properly define the accuracy of the ground control points/initial exterior orientation (EO) parameters when they are derived from GNSS/IMU measurements. Proper definition of the camera calibration parameters and the accuracy of the initial EOs and ground control point coordinates are crucial for achieving accurate AT solutions.

Additionally, AT software packages often offer camera self-calibration options, which can be dangerous if used on datasets which do not provide sufficient geometry for adjusting certain parameters. For example, camera focal length should only be adjusted when the dataset includes flight lines flown at multiple altitudes, especially when mapping flat terrain.

When performing aerial triangulation, an independent, unconstrained adjustment should first be run to determine if there are any problems with the ground control, GNSS measurements, and/or image measurements. Any issues with these measurements, including datum problems, should be identified and resolved. After resolving these issues, a constrained adjustment can then be run.

SECTION I: ACCEPTANCE CRITERIA FOR AERIAL TRIANGULATION RESULTS

The aerial triangulation and/or GNSS/IMU-based direct georeferencing accuracies must be of higher accuracy than the required accuracy of the final product.

To evaluate the accuracy of the aerial triangulation, the root-mean-square error (RMSE) of checkpoints should be computed by comparing the checkpoints' coordinate values (as calculated in the aerial triangulation solution) to their surveyed coordinates. The $RMSE_X$, $RMSE_Y$, and $RMSE_Z$ of the checkpoints should then be computed.

Alternatively, the $RMSE_X$, $RMSE_Y$, and $RMSE_Z$ of the checkpoints can also be computed by comparing the coordinates read from the imagery (using stereo photogrammetric measurements or another appropriate method) to the surveyed coordinates of the checkpoints.

For projects that require orthoimagery and/or digital planimetric map delivery only, the ASPRS Accuracy Standards define the following aerial triangulation acceptance criteria:

$$\mathrm{RMSE}_{\mathrm{H(AT)}} \leq ½ * \mathrm{RMSE}_{\mathrm{H(MAP)}}$$

$$\mathrm{RMSE}_{\mathrm{V(AT)}} \leq \mathrm{RMSE}_{\mathrm{H(MAP)}}$$

where:

$$\mathrm{RMSE}_{\mathrm{H(AT)}} = \sqrt{\mathrm{RMSE}_{\mathrm{X}}^2 + \mathrm{RMSE}_{\mathrm{Y}}^2}$$

Accuracy of aerial triangulation designed for elevation data production or planimetric data (such as orthoimagery and/or digital planimetric mapping) and elevation data production should meet the following accuracy criteria:

$$\mathrm{RMSE}_{\mathrm{H(AT)}} \leq ½ * \mathrm{RMSE}_{\mathrm{H(MAP)}}$$

$$\mathrm{RMSE}_{\mathrm{V(AT)}} \leq ½ * \mathrm{RMSE}_{\mathrm{V(DEM)}}$$

Currently, the ASPRS Accuracy Standards assume that the GCP RMSE also has to meet the same criteria. The number of GCPs depends on the block geometry, intended product accuracy, and airborne GNSS quality. AT results and accuracy should be included in the project report and metadata.

When a checkpoint or control point residual in either X, Y, or Z exceeds three times the required RMSE threshold, the checkpoint or control point should be considered a blunder. Blunder checkpoints should not be discarded without proper investigation. If the source of the error can be identified, then it should be corrected if possible, and the checkpoint should be included in the final accuracy reporting. If the error cannot be corrected, then the blunder checkpoint can be excluded from the final accuracy computation and reporting only after proper explanation.

In some cases, users of the ASPRS Accuracy Standards may find it difficult to meet the above aerial triangulation accuracy requirements, especially now that ground control points and checkpoints are only required to be twice as accurate as the final product. In cases where the aerial triangulation results do not pass the RMSE criteria, but do meet the RMSE requirements of the final product, the data producer and the data user may negotiate and determine if the aerial triangulation accuracy is acceptable.

Meeting the checkpoint and control point aerial triangulation RMSE criteria alone does not guarantee that the aerial triangulation results are valid. To increase confidence in the aerial triangulation results, the tie point coverage and distribution of the residuals (including Von Gruber areas) should also be evaluated. There should be no gaps in tie point coverage. The tie point residuals should be randomly oriented, and the RMSE of image coordinate residuals should be within 1 pixel.

In case of GNSS/IMU-supported AT where initial EOs were derived from the airborne GNSS/IMU solution, it is important to evaluate how much the AT solution changed the initial EOs, in addition to the checks described above. The changes in the initial EOs should be commensurate with the expected accuracy of the GNSS/IMU solution. If the changes are significantly larger than expected, then this should be investigated. Depending on the result, the AT may need to be rerun with a modified initial EO accuracy input.

In the case of direct georeferencing, when image exterior orientation parameters are determined by GNSS/IMU measurements and no aerial triangulation is performed, the $RMSE_X$, $RMSE_Y$, and $RMSE_Z$ of the checkpoints can be computed by comparing the coordinates read from the imagery (using stereo photogrammetric measurements or another appropriate method) to the surveyed coordinates of the checkpoints. The computed RMSE values should meet the same accuracy as listed for the aerial triangulation acceptance criteria.

SECTION J: AERIAL TRIANGULATION ACCURACY

Aerial triangulation accuracy depends on many interrelated factors, including:

- Flying altitude
- Camera calibration parameter quality
- Quality and size of the CCD or CMOS used in the digital camera sensor array
- Amount of imagery overlap
- Quality of parallax determination or photo measurements
- Quality of the GNSS signal
- Quality and density of ground control points
- Quality of the aerial triangulation solution
- Capability of the processing software to handle GNSS drift and shift
- Capability of the processing software to handle camera self-calibration
- The digital terrain model used to produce the orthoimagery

In general, aerial triangulation accuracy $\mathrm{RMSE_{AT}}$ can be expressed as:

$$\mathrm{RMSE_{AT}} = \frac{\mathrm{H}}{\mathrm{factor}}$$

Where H is the flying height, and factor is a constant that varies between 2000 and 15000 for vertical accuracy and 2000 and 20000 for planimetric accuracy. The factor is a function of the parameters listed earlier. This factor should be evaluated for the camera system using a control field with at least 30 checkpoints. This formula will help design the photogrammetric block to achieve the required aerial triangulation accuracy.

When a non-metric camera is being used (as in UAS), camera self-calibration is highly recommended. If the terrain is flat, multiple flying heights should be used.

SECTION K: DIRECT GEOREFERENCING

Direct georeferencing is the direct measurement of the position and orientation parameters of a sensor. In photogrammetry, direct georeferencing can be used to produce measurements of the exterior orientation parameters for each image without the use of ground control and aerial triangulation. This may be achieved by using data from an onboard GNSS receiver (or receivers) and an IMU capable of determining the position and orientation of the aircraft during the image acquisition process. This process involves collecting GNSS/IMU data during the flight, synchronizing the GNSS/IMU data with the sensor data through precise timing, and then using these measurements to georeference the images.

The first step is to process the GNSS/IMU data. This processing is performed using hardware-specific software. There are various GNSS/IMU processing techniques in use. Some techniques use one or more base stations for differential GNSS processing, while others require a subscription service to provide correction parameters. Precise Point Positioning (PPP) may be another option, depending on the required accuracy. Processing methods can also be categorized in terms of loosely- or tightly-coupled processing. The data producer should select an appropriate processing technique that can support the desired accuracy of the product.

Many details need to be considered during GNSS/IMU processing. The data producer should ensure that the correct GNSS lever arms are applied, and that IMU mounting angles and lever arms are properly defined. Most GNSS/IMU processing software allows for the estimation of the GNSS lever arms, and it is good practice to double-check and, in some cases, refine the known GNSS lever arms before they are used to process the airborne trajectory.

The GNSS/IMU solution should be carefully checked for accuracy. Depending on the software used, there are various plots that can be reviewed to ensure accuracy. The accuracy of the final GNSS/IMU solution should be better than the required accuracy of the product.

When generating exterior orientation parameters for the images, the correct camera boresight angles and camera lever arms should be used. The camera lever arms define the relative position of the camera with respect to the reference point to which the GNSS/IMU solution was processed. These lever arms are typically calibrated by the sensor manufacturer to a high level of accuracy. The physical misalignment between the IMU body frame and the camera frame is described by the camera boresight angles. The boresight angles have to be calibrated via a calibration flight. A specific calibration flight pattern is required to solve the camera boresight angles using a least-squares bundle adjustment. When using direct georeferencing, it is critical to keep the camera boresight angles up to date, because any error in the boresight angles will affect the accuracy of the derived products—the higher the flying altitude, the larger the error in the derived product will be. The boresight angles should be recalibrated at regular intervals, according to the sensor manufacturer's recommendations.

When generating EOs, the exterior orientation angles (ω, ϕ, κ) can be output using the desired order of rotations. Along with the exterior orientation information, it is critical to provide the order of rotations so that the EOs can be used correctly in photogrammetry software; if the order of rotations is not correctly defined in the photogrammetry software when importing the EO parameters, serious accuracy issues will occur. Data users should check their photogrammetry software's user manual to verify the required order of rotations.

SECTION L: DIGITAL SURFACE MODEL GENERATION

Digital surface model (DSM) generation from photogrammetry entails creating a "first visible" product of features (i.e., features that are first visible from the air, such as trees and buildings). This includes ground in open areas with no above-ground features. While DSMs can be created manually via stereo compilation, this approach is labor intensive. Methods for automatically generating DSMs vary, but all incorporate forms of image matching, where coarse-to-fine image matching algorithms identify either matching points or matching features (such as points and edges). It is important to note that most automated or semi-automated processes usually require some level of quality control, depending upon the project requirements.

The method for producing photogrammetrically-derived DSMs should be chosen according to the purpose of the final product. The following methods, along with their strengths and weaknesses, are described below:

- *Manually Generated* – rarely used, generated utilizing manual stereo compilation methods. Highly accurate and suitable for obstruction mapping and 3D modeling. Requires little QA/QC, but is very labor intensive.
- *Structure from Motion (SfM)* – fully automated, with potential for matching errors. Since this is usually run without ground control coordinates, the camera positions derived from SfM lack scale and orientation. This results in 3D point clouds residing in an image-space coordinate system, which must be post-processed to an object-space coordinate system. May require significant manual QA/QC.
- *Autocorrelated DSM w/ Control* **–** fully or semi-automated. This approach assumes use of data that has undergone an aerial triangulation process utilizing ground control. This eliminates the need to post-process the DSM to an object-space environment and may provide greater flexibility in post-processing options. May require some manual QA/QC.

SECTION M: ORTHOPHOTO GENERATION

An orthophoto is a product that has both the pictorial qualities of a photograph and the planimetric correctness of a map. An orthophoto shows images of objects in their true orthographic positions. Orthophotos are produced via differential rectification, which eliminates image displacements due to photographic tilt and terrain relief.

For conventional ground orthophoto generation, the required inputs are: the final image EO parameters, the camera calibration information, the DEM, and the radiometrically-corrected imagery.

M.1 DEM Sources

A DEM can be derived from the auto-correlated DSM that was generated from the imagery by removing above-ground features from the DSM. This can be done by filtering using a macro optimized for the terrain type. Additional manual editing may be required to place bridges in the data or remove noise. If lidar data was collected along with the imagery, then it is recommended that the lidar-derived DEM be used instead. A DEM can also be derived from a preexisting, outside source such as lidar, stereo compilation, etc., as long as the accuracy and resolution of the source data is sufficient.

If the DEM used for orthorectification does not represent the ground surface accurately, significant distortions may occur in the derived orthoimagery. Table III.M.1 shows the error in the derived orthoimagery as a function of error in the DEM and the field of view off nadir. At nadir, DEM error has no effect on the orthoimagery, but the greater the distance a pixel is from nadir, the higher the effect of DEM error.

Table III.M.1 Effect of DEM Error on Orthoimagery

Error in DEM [m]	Error in Orthos at nadir [m]	Error in Orthos at 7.5 degrees off nadir [m]	Error in Orthos at 15 degrees off nadir [m]	Error in Orthos at 30 degrees off nadir [m]	Error in Orthos at 45 degrees off nadir [m]
0.10	0.00	0.01	0.03	0.06	0.10
0.20	0.00	0.03	0.05	0.12	0.20
0.50	0.00	0.07	0.13	0.29	0.50
1.00	0.00	0.13	0.27	0.58	1.00
2.50	0.00	0.33	0.67	1.44	2.50
5.00	0.00	0.66	1.34	2.89	5.00
10.00	0.00	1.32	2.68	5.77	10.00

M.2 Source Imagery

According to these Standards, the GSD acquired by the sensor (and as computed at mean average terrain) should not be more than 95% of the final orthoimage pixel size when producing orthoimagery. In extremely steep terrain, additional consideration may need to be given to the variation of the GSD across low-lying areas to ensure that the variation in GSD across the entire image does not significantly exceed the target pixel size. The USGS Digital Orthoimagery Base Specifications v1.0, Table 1 also provides guidelines on the acceptable minimum and maximum GSD of the source imagery depending on the required orthoimagery pixel size.

M.3 Orthorectification Process

The orthorectification process is performed using photogrammetric processing software. This process generally requires the camera calibration information, the DEM, the source imagery, and the final image EO parameters that are typically generated after aerial triangulation. The process of orthophoto generation requires the repetitive application of the collinearity equations for all the points in the DEM array, which involves a resampling process. Most photogrammetry software packages offer multiple resampling processes to choose from, though cubic convolution is often preferred over other options like nearest-neighbor or bilinear interpolation.

Once the rectification process is complete, the analyst should ensure that the process ran successfully and that the appropriate coverage is fulfilled. To ensure quality and accuracy of the orthoimagery, the analyst should QC multiple orthophoto samples covering the project area. Orthophoto samples covering the areas surrounding the checkpoints are used to assess the absolute orthophoto accuracy, while orthophotos between adjacent images are used to assess the relative orthophoto accuracy.

M.4 Orthophoto Mosaicking

After the imagery has been orthorectified and quality-checked, the mosaicking process can begin. Automated seamlines are generated in the image overlap areas. The analyst should review the automated seamlines and modify the placement as necessary to improve the quality of the join area in the output once mosaicked. The results should create a seamless final product. As stated in the USGS Digital Orthoimagery Base Specifications v1.0, when selecting seamlines, specular reflections and other artifacts should be minimized (especially in developed areas) by using chips from adjacent overlapping imagery. Above-ground features appearing in the orthoimagery, such as building rooftops, water towers, and radio towers, should not be clipped at seamlines.

Radiometric balancing of the orthoimagery should also be done to ensure that the brightness and color of the orthomosaic is consistent throughout. The seamlines between overlapping images should be chosen to minimize tonal variations. Localized adjustment of the brightness and color values should be done to reduce radiometric differences between join areas.

M.5 Accuracy Assessment

A final QA/QC should be performed on the orthoimagery mosaic to ensure that it meets the required accuracy specifications. It is recommended that the final QA/QC be done by an analyst other than the one who worked on the orthoimagery mosaic generation. The horizontal positional accuracy of the orthoimagery mosaic should be evaluated using checkpoints. For testing orthoimagery, checkpoints should not be selected on features elevated with respect to the DEM used to rectify the imagery. The orthoimagery mosaic should also be evaluated for seamline mismatch.

The computed horizontal positional accuracy, expressed as $RMSE_H$, should be less than or equal to the specified ASPRS horizontal accuracy class. It is important to note that $RMSE_H$ also factors in the checkpoint survey accuracy, as described in section 7.11 of these Standards. In addition to the horizontal positioning accuracy, the orthoimagery mosaic must also be evaluated for seamline mismatch. As described in section 7.3 of these Standards, the maximum allowable seamline mismatch is $\leq 2 * RMSE_H$.

M.6 True Orthophoto

A "true" orthophoto is an orthophoto that has every pixel placed in its "true" geographical location, including all above-ground features such as buildings or trees (as opposed to a conventional orthophoto where only on-the-ground features are in their true geographical location). True orthophotos are most requested for heavy urban areas where tall buildings can obscure the sidewalks and roads next to them due to building lean present in a conventional orthophoto.

Here are some options currently offered in the marketplace for true orthophoto generation:

- True orthophotos generated from high-resolution DSM using images with very high endlap and sidelap
- True orthophotos from SfM-generated, high-resolution 3D models

- Reduced building lean or "near" true orthophotos where additional imagery is collected, the imagery is orthorectified using a ground-based DEM, and extensive seamline editing is employed to use the most nadir view of each building in the final mosaic

SECTION N: ACCURACY TESTING AND REPORTING FOR PHOTOGRAMMETRIC PROJECTS

This section outlines recommended best practices and guidelines for accuracy testing, photogrammetric project reporting, and accuracy reporting. This section is not intended to fully outline processes and guidelines such as analysis formulas or number of checkpoints over a given area. The following subsections on accuracy testing and reporting are generalized guidelines. For more detailed information, please refer to the ASPRS Accuracy Standards.

Entities such as those at the federal or state level may require testing and/or project documentation to include reporting on processing workflows and accuracy results. It is recommended that, in lieu of any contractual or specified requirement, projects utilizing photogrammetry still document accuracies for products and report on data acquisition and processing efforts. This ensures that the products and workflows are documented, regardless of when or where the data may be used. This may also help prevent photogrammetric data from being used for purposes that it may not support; for example, a data user attempting to utilize photogrammetric data to generate accurate 0.25-ft contours from a photogrammetrically-derived DTM produced to support a resolution of 1-ft contours.

N.1 General Reporting

The project reports should include the components below, as well as any additional elements according to the project requirements. These components are sometimes combined into a single project report, or they may exist in separate reports depending on the project stakeholders and/or project specifications being used. The exact language or level of detail may vary according to project specifications and requirements.

- *Aerial Collection Report* – should contain as-flown information such as flight plan, flight logs, dates of flight, sensor used, sensor settings, initial QA/QC, problems encountered, and lessons learned (if any).
- *Ground Survey Report* – should be written and sealed by a licensed surveyor, and should contain a map of locations, survey procedures used, the horizontal and vertical datum, process used to verify the accuracy of the GNSS observations (along with results), a listing of any CORSs used, data sheets for points used in the aerial triangulation and/or points used for independent verification or QC, any sketches or photos, problems encountered, and lessons learned (if any).
- *Processing Report* – should incorporate an aerial triangulation report, and generally includes a description of the processes used during data/product development, QA/QC procedures, descriptions of the products, problems encountered, and lessons learned (if any).
- *Accuracy Statement/Report* – should utilize either the "tested to meet" or "produced to meet" statements when describing the accuracy of a product. The methodology used to test, as well as

the results, should be included for "tested to meet" reports. These are described in more detail later in this section.

With the exception of the land surveyor's report, reports should be reviewed and sealed by an ASPRS-certified photogrammetrist. This ensures proper review and establishes that a qualified individual has certified the accuracy of the information provided. Consult the ASPRS website for certification options.

N.2 Accuracy Testing and Reporting

The processes used to test the accuracy of photogrammetrically-derived products should ensure that the product meets or exceeds the minimum accuracy criteria outlined in the project specifications for each product. Accuracy criteria may be established by published standards and guidelines such as the ASPRS Accuracy Standards. The organization commissioning the project may also have their own standards that the products must be tested against; for the purpose of this Addendum, it will be assumed that the specifications of the project are aligned with the ASPRS Accuracy Standards.

Accuracy statements fall into one of the following categories:

- "Tested to meet" – implies that well-defined points in the product are compared with coordinates (X, Y and/or Z) determined from an independent source of higher accuracy.
- "Produced to meet" – implies that the project design (sensor, sensor settings, flying height, control, etc.), quality of the equipment, and processes used all support the accuracy requirements of the project, but the product has not been tested per the "tested to meet" guidelines.

N.3 Accuracy Testing – "Tested to Meet"

When testing a photogrammetric product, the data of higher accuracy, such as ground survey checkpoints, should be truly independent of the photogrammetric processes used. For example, a set of ground survey checkpoints reserved for testing the product should not be used as control in the aerial triangulation, nor should they be used at any point in the photogrammetric process except for testing the final product and reporting on the accuracy.

To support a "tested to meet" statement, the coordinates of higher accuracy used to test the product should be well-distributed across the geographic extents of the product if possible, or representative of the features in the product being tested. For example, testing the horizontal accuracy of planimetric features in a photogrammetric map may only be possible where these features exist, but testing the vertical accuracy of a photogrammetrically-derived DTM is likely possible throughout the extent of the product.

It is important to have a sufficient number of coordinates from the data of higher accuracy in order to conduct a statistically robust analysis. At a minimum, 30 coordinates of higher accuracy should be used, as this provides a good foundation for statistical analysis.

Ethical considerations for accuracy testing can be summarized by these recommended guidelines:

- Do not use the control data (data of higher accuracy) in the photogrammetric workflow of a product if you are using the data to independently test the product.

- Do not eliminate checkpoint coordinates that return a residual higher than the accuracy threshold without a valid reason to eliminate the coordinate, such as a bad survey or improper placement of the checkpoint. Any eliminated checkpoints should be well-documented in the report.

If the results barely meet the accuracy threshold, this may be an indication of error or bias somewhere in the photogrammetric workflow. In such a scenario, there is a greater chance of the product not meeting specifications in untested geographic areas. This bears further investigation by the data producer.

Geographic areas within the product that may not meet accuracy thresholds for reasons which are out of control of the data producer should be delineated using a method that clearly communicates this to the data user, such as obscured area polygons. Planimetric and terrain features obscured from the aerial view by shadows or vegetation are common reasons to delineate such areas. Akin to the purpose of dashed contours, digital linework delineating such areas is an important component for communicating product accuracy.

The current ASPRS Accuracy Standards also require factoring in the accuracy of the checkpoints when reporting the tested horizontal, vertical, and three-dimensional positional accuracy. Accuracy should be reported as described in section 7.16 of the ASPRS Accuracy Standards.

N.4 Accuracy Testing and Reporting – "Produced to Meet"

When the products are not tested using an independent source of higher accuracy, the data producer must certify that the project design, equipment, processes, and QA/QC methodologies support the accuracy specifications of the project. For this type of reporting, the information provided should include (but is not limited to):

- Project design information – as-flown information, survey control plan, and sensor settings
- Equipment – lists and/or descriptions of the equipment used, such as aircraft, sensor, IMU, airborne GPS, and production hardware
- Software – lists and/or descriptions of the software used throughout the workflow
- Production processes – generalized descriptions of the processes used to derive a given product
- Quality processes – quality steps undertaken during each phase of the photogrammetric workflow

As with the guidelines provided in the "tested to meet" section, geographic areas within a given product that may not meet specifications due to being obscured should be delineated and communicated to the data user.

Accuracy reporting should be performed as described in section 7.16 of these Standards.

ADDENDUM IV: GUIDELINES AND BEST PRACTICES FOR MAPPING WITH LIDAR

Contents

FIGURES

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PURPOSE

The purpose of this Addendum is to provide general best practices and guidelines for airborne topographic lidar production workflows, with a focus on positional accuracy. These guidelines are intended to be sensor- and manufacturer-agnostic.

The intent of this Addendum is to establish a resource for airborne topographic lidar data producers which describes current industry best practices and guidelines for airborne topographic lidar data production workflow. In addition, this Addendum is also intended to assist data users when defining lidar data collection and processing requirements, according to the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024), referred to here as the ASPRS Accuracy Standards.

This Addendum will also cover best practices for project planning, ground control point (GCP) and checkpoint planning, data acquisition, lidar data processing, and accuracy assessment and reporting. This Addendum is not meant to substitute or override sensor and software manufacturer manuals and recommendations. Additionally, this Addendum will assume that the reader is experienced with the lidar production workflow; it is not intended to provide instruction on how to perform lidar production workflow tasks.

SECTION A: LIDAR FLIGHT PLANNING

A.1 Introduction

Well-designed flight plans are critical to ensure flight safety, completion of contract requirements, and efficient data collection.

Effective lidar flight planning requires balancing several variables to optimize sensor performance and flight time in order to meet project requirements. Variables include Signal to Noise (SNR) ratio, aircraft speed, Field of View (FOV), cross-track/down-track or along-track point spacing (symmetry), flying height, Pulse Repetition Frequency (PRF), Multiple Pulses in Air (MPIA), scan pattern, flight line orientation, and scan frequency/scan rate. Faster flight speeds do not always reduce flying time, as flight speed must take sensor efficiency into account during flight planning.

Other factors such as airspace restrictions, sensor performance, and aircraft performance must also be taken into account. It may be beneficial for the flight planner to be able to read aviation sectional charts, depending upon the airspace in which the project occurs.

The flight planner must carefully weigh his or her options, as the flight plan has a direct impact on project cost and data quality and should therefore be crafted with care.

A.2 Overview

A.2.1 Software

Lidar flight planning software is usually provided by the sensor manufacturer. This software is often proprietary, but commercially available software packages do exist and can be configured to operate with some sensors.

For large-scale operations, it may be necessary to closely manage compatibility between flight planning software and the project’s Flight Management System (FMS). Firmware upgrades to sensors may require contemporaneous flight planning software and FMS upgrades. Therefore, it may be useful to include the version of the planning software in the name of the flight plan.

A.2.2 Project Specifications

Flight plans should be designed to meet project specifications. The flight planner should be familiar with the requirements set forth in the contract and with the goals of the project, including the required accuracy, the desired point density, the intended applications of the collected data, etc. In some cases, there may be additional factors to consider during flight planning, such as maximum FOV or maximum footprint size requirements. A power line mapping project, for example, may require a different planning approach than a forestry project.

A.2.3 Airspace

When planning flight lines, it is important to take into account air traffic conditions and airspace restrictions. If planning for collection in busy airspace, the flight planner may ask Air Traffic Control (ATC) for input on when to collect data, what altitudes would be ideal, and appropriate flight line orientations.

For restricted airspace, the flight planner should seek prior authorization; until this authorization is obtained, flight lines should never cross into restricted airspace. Flight lines should be either oriented parallel to the restricted area or else terminated at an adequate distance to allow the aircraft to turn before entering the restricted airspace.

A.2.4 Aircraft

The flight plan should take into account the type of aircraft (i.e., rotary wing versus fixed wing), as well as the performance of the aircraft. For example, if the collection speed is 160 knots in the flight plan, but the aircraft cannot achieve this, then the data collection will take more time than estimated and the aircraft may end up exceeding the recommended maximum time on-line limit.

A.2.5 Terrain

When mapping mountainous terrain, flight lines should run parallel to mountains and tall ridges. This is necessary to optimize the performance of the sensor, and to ensure safety during operation.

Many lidar sensors which handle multiple points in the air operate more efficiently at a consistent Above Ground Level (AGL), so mitigating extreme fluctuations in terrain height (such as those caused by mountains and ridges) will allow for more efficient sensor operation. The sensor's Field of View (FOV) may also be affected by changes in terrain height, though some sensors are designed to account for this.

Regardless of sensor capability and performance, terrain presents a potential hazard to crew safety, which can be mitigated by flying parallel to terrain features such as mountains and ridges. The planner should be aware of such terrain features and should account for them when determining flight line position and direction. Terrain should be taken into account when planning turns, so as to not put the flight crew in danger—for this reason, it is very important to be aware of the aircraft's turn radius and vertical clearance needs. Most flight planning software includes a digital terrain model and a way to preview potential flight lines in profile over ground. The planner should also have access to a pilot to address issues about safety.

A.2.6 Corridors

Planning for collection along corridors from a fixed wing aircraft generally requires decisions related to time spent in turns versus time spent flying straight over sections outside the Area of Interest (AOI) to pick up another line section. The planner should have some knowledge about the turn time of the aircraft in order to make these decisions. It often saves time for fixed wing aircraft to fly a corridor in blocks of lines along sections of the corridor, rather than starting a new line at each turn in the corridor.

A.2.7 Flight Line Numbering

Flight lines should be numbered consecutively, following the geographic order mapped out in the flight plan. For multiblock projects, many data producers use different number sequences for each block—for example, flight lines in block 1 may start at line 101, while flight lines in block 2 may start at line 201. This can help avoid confusion during data collection and processing, and can help ensure clear communication with ATC by allowing them to locate or reference specific flight lines more efficiently. For large blocks, it may be necessary to start in the thousands rather than the hundreds (i.e., starting block 1 at line 1001).

A.2.8 Blocks

Blocks are an easy way to break larger projects up into more manageable subsections. Blocks are usually small geographic subsets or regions of a larger AOI. Depending on the processing workflow, blocks of flight lines may be oriented independently to allow more flexibility when planning around terrain. Blocks can be particularly useful if deploying numerous aircraft on the same project, as individual blocks can be assigned to each aircraft.

A.2.9 Differential Corrections

In the event base stations or CORS are to be used for post processing, consideration should be given to the maximum distance from the correction source. This distance will vary by manufacturer, hardware version (make and model), and post-processing software, or it may be stipulated in the contract based on the client's requirements. The location of the correction source should be provided to the sensor operator and/or flight crew for Global Navigation Satellite System (GNSS) initialization.

A.2.10 Terrain Flying versus Straight and Level

If used correctly, terrain flying (or terrain following) may reduce flight times, but it may not work well with certain sensors or multi-sensor platforms. In certain terrain types, terrain flying may present a hazard to the crew; in these cases, the safety of the crew should be prioritized.

SECTION B: LIDAR PLANNING VARIABLES

B.1 Point Density

Point density is a critical component of all lidar projects. It is measured by points per square meter (ppsm) and is affected by factors such as: PRF; scan rate; the aircraft's groundspeed; the scanner's FOV; Above Ground Level (AGL); flight line sidelap; and how many sensors, channels, and/or planned passes are used.

Since the density of an unstructured and unorganized point cloud may vary, point density must be considered a distribution rather than a single number. If lidar acquisition is planned for the minimum required density, it is likely that a significant portion of the acquired data will fail an appropriate density test. In the example below, if a user plans for an average design density (blue line) of 8 ppsm, and the project requirement is 8 ppsm (meaning a required minimum of 8 ppsm), approximately half the data could be <8 ppsm (red) and the other half > 8 ppsm (green), as depicted in Figure IV.B.1a. If the flight planning takes this into consideration and compensates accordingly, most of the data will fall >8 ppsm, as depicted in Figure IV.B.1b.

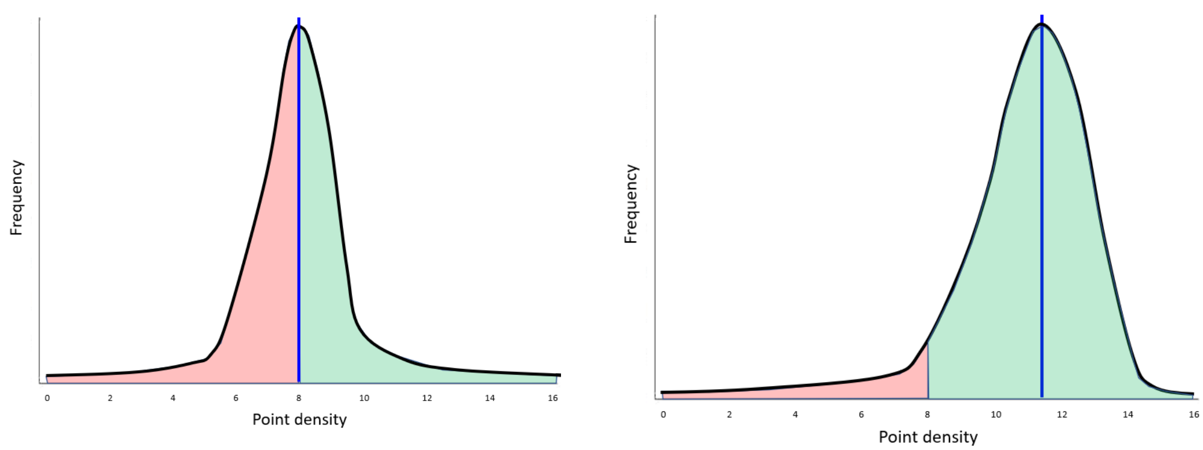

Figure IV.B.1 Point density distribution curves. a) Point density planned for average design density and b) point density planned for minimum required density.

B.2 Signal to Noise Ratio

The Signal to Noise Ratio (SNR) is related to how much energy the laser of the lidar sensor emits per pulse and the reflectance of the target on the ground. A ratio greater than 1 indicates that there is more signal than noise. Generally, as the Pulse Repetition Frequency (PRF) increases, the SNR decreases; in other words, increasing the PRF will result in faster data acquisition at the cost of increased signal noise, potentially reducing precision.

If the SNR is very low, this could indicate that some targets are not being detected adequately. Therefore, the planner must understand the goals of the project and the ability of the sensor to detect certain targets at different SNRs.

B.3 Scan Pattern

Modern lidar sensors often support more than one scan pattern. Common patterns are circular, parallel line, sawtooth (zig-zag), and sinusoidal:

- A circular pattern sweeps out and behind the senor in an elliptical or circular pattern. This tends to allow for returns off the sides of buildings.
- The parallel line pattern appears as a series of parallel lines on the ground.
- The sinusoidal or zig-zag pattern looks like a series of connected lines that cross back and forth within the swath. The pattern may have either sharp or slightly rounded inflections at the edge of the swath.

The planner should understand the strengths, weaknesses, and consequences of different scan patterns. For a circular pattern, for example, the planner may extend the end of the flight lines past the AOI so the rear sweeping scan passes through the AOI at the end of the flight line to help maintain point density.

B.4 Point Distribution

The distance between laser returns in the cross-track direction (perpendicular to flight direction) and the down-track/along-track direction (parallel to flight direction) should be similar. Some variations in cross-track and down-track resolution are likely to occur due to changes in aircraft speed and yaw during data collection. However, the flight plan should always strive to achieve symmetry, as reduced symmetry may impede target detection and could result in data not meeting contract requirements.

B.5 Swath Overlap

Overlapping adjacent swaths serve several purposes:

- Increases point density (some flight plans achieve required point density via double coverage)
- Reduces “laser shadows” by hitting targets from multiple look angles
- Reduces the risk of gaps in data due to turbulence
- Aids in boresight and/or flight line adjustments

Additionally, some sensors experience edge artifacts, such as increased noise at the edge of swaths. For this reason, it is best practice to “clip” or reclassify noise at the edge of swaths during processing. The flight plan should provide sufficient overlap to accommodate clipping, if necessary.

B.6 Length of Flight lines

If flight lines are too long, they may begin to induce drift in the Inertial Measurement Unit (IMU). Different makes and models of GNSS-assisted Inertial Navigation Systems have different tolerances for straight and level flight before acceptable solution tolerances are exceeded. Follow the manufacturer's recommendations on maximum time on-line limit.

Drift can manifest as a relative offset in the data. This is particularly easy to see (and quantify) in lidar due to overlap, precision, and density of the data, as well as the fact that overlapping points are independent of each other.

B.7 Multiple Pulses in Air (MPIA)

Multiple Pulses in Air (MPIA) help achieve higher point densities and may reduce collection times. This technology may have limited efficacy if there are many fluctuations in distance from the sensor to the ground while on-line. Flight planning software can help identify potential issues related to too many points in the air over uneven terrain.

Note that some vertical deviation from planned collection altitude is inevitable. Planning within overly tight margins may result in re-flights due to dropped returns.

B.8 Flight line Orientation

Flight line orientation should be optimized to reduce collection times. Airspace, terrain, and data processing workflows should be taken into account when planning orientation.

B.9 Field of View

Field of View (FOV) affects the width of the swath on the ground. For fixed FOV sensors, a decrease in AGL narrows the width of the swath on the ground. Flight lines should be distanced such that project requirements will be met even at the narrowest FOV. This may be augmented with additional lines. Some sensors have a dynamically adjustable FOV that provides a constant swath width across fluctuations in terrain height.

B.10 Eye Safety

Lidar flight lines should be flown above the minimum threshold for eye-safe operation. The Nominal Ocular Hazard Distance (NOHD) and the Extended Nominal Ocular Hazard Distance (ENOHD) should be readily available information obtainable from the sensor manufacturer. This distance is unique to different makes and models of lidar systems.

B.11 Cross Lines

Cross lines are flown perpendicular to the orientation of a block of lines used in production. There may be one line across the middle of a block, or one at each end. They provide a common reference for flight line adjustment and aid in boresight angle calculations.

B.12 Flight Line Direction

Consecutive neighboring flight lines should be flown in opposing directions. This aids in post-processing when solving for boresight, and allows the craft to fly more efficiently by turning at the end of one line into the start of the next line.

B.13 Tolerances

The maximum acceptable deviation from planned collection specifications should be determined prior to data acquisition. This is typically a set of standards that varies by vendor. Variables such as maximum acceptable deviation from planned altitude and collection speed, as well as maximum distance from line, should all be determined prior to collection. When tolerances are exceeded, the data should be recollected and noted on the flight log. This may include part or all of a line.

Navigation solution tolerances (the ability to integrate GNSS and IMU data in real time) are often set by the sensor manufacturer and should be periodically verified by skilled professionals. In cases where the hardware is new to the user, navigation solution tolerances should be checked prior to any collection attempts. Data producers should follow manufacturer recommendations for settings, as these may vary depending on the make and model of the trajectory hardware.

B.14 Planning Ground Control

After the flight plan is complete, GCPs and checkpoints should be planned out. The planner should coordinate this with the project manager and/or survey team ahead of time, to ensure an understanding of the requirements of the project and to determine any pertinent limitations that may need to be considered. The number of GCPs and checkpoints should meet the needs of the project and should adhere to the minimum requirements of the ASPRS Accuracy Standards. GCPs should be evenly placed throughout the project area where possible. It is recommended to place a few GCPs on cross line extents to achieve better accuracy. Details about the ground survey are provided in the next section.

SECTION C: GROUND CONTROL POINTS AND CHECKPOINTS

C.1 Introduction

Ground control points (GCPs) are used for lidar calibration, while checkpoints are used when determining the accuracy of lidar data. Checkpoints are further classified as Non-Vegetated Vertical Accuracy (NVA) and Vegetated Vertical Accuracy (VVA) points.

GNSS surveying is the most commonly-used approach to establish control points for aerial lidar survey projects. Hence, in this document, best practices and guidelines for establishing GCPs via GNSS surveying will be described.

C.2 Background

The ASPRS Accuracy Standards address geolocation accuracies of geospatial products. This document describes positional accuracy standards for digital orthoimagery, digital planimetric data, and digital elevation data. This document does not specify the best practices or methodologies needed to meet the accuracy thresholds stated herein.

C.3 Licensed Surveyor

The ground survey to establish the GCPs and checkpoints should be conducted under the direct supervision of a licensed land surveyor (or equivalent within that state). Every state has established rules and regulations related to conducting GNSS surveys within their jurisdictions. These rules should be investigated and followed before conducting ground surveys.

C.4 Number and Distribution of Ground Control Points and Checkpoints

The locations of ground control points are discussed in this section. The control points should be located around the border and in the center of a project area.

Control points should be evenly distributed over the project area. Site access may be a factor in some locations, but every effort should be made to distribute control points throughout the project. Distribution helps account for small spatial variations in bias due to GNSS errors and ranging errors. GCPs should be sufficient in quantity to debias the lidar data.

Appendix C of the ASPRS Accuracy Standards describes the requirements for the minimum number of checkpoints based on project area. It also addresses the minimum required number of NVA and VVA points. The number of VVA points should be divided proportionally among each of the major vegetation cover categories.

C.5 Ground Surface Conditions

This section describes the ground surface conditions where the GCPs, NVAs, and VVAs should be located in a project.

C.5.1 Control and NVA Points

When selecting locations for control and NVA points, the following factors should be considered:

1. Vertical checkpoints are not required to be clearly defined or located on visibly identifiable point features, unlike horizontal checkpoints.
2. Vertical checkpoints shall be surveyed on flat or uniformly sloped open terrain with slopes of 10% or less. These checkpoints should avoid vertical artifacts or abrupt changes in elevation.
3. Control and NVA points shall be established on flat surfaces, such as sidewalks and road surfaces with less than 10% slope. Some suitable locations include paved surfaces and firmly packed dirt. Figure IV.C.1 shows two examples of good locations for a lidar control or NVA point. Note that the ground surface is flat in both examples. Additionally, the location is away from the curb or any other features with abrupt changes in elevation. These points are also away from crosswalk or transportation markings.

Figure IV.C.1 Preferred ground surface conditions for GCPs and checkpoints.

4. Figure IV.C.2 shows four examples of bad locations for a lidar control or NVA point. These points are on ground that is not flat and which has changes in elevation close to the observed point. Vertical checkpoints should be established at locations that minimize errors when comparing elevations interpolated from the data set to the elevations of the checkpoints.

Figure IV.C.2 Locations that are not appropriate for GCPs and checkpoints.

5. Locations that have a high likelihood of being obstructed during lidar data collection (for example parking spots) should be avoided.
6. Points shall be selected to minimize interpolation errors when comparing elevations interpolated from the data set to the elevations of the checkpoints.
7. As a rule of thumb, an inverted cone of θ = not more than 30–45 degrees at 1 meter distance from the point should be free of any obstruction (Figure IV.C.3).

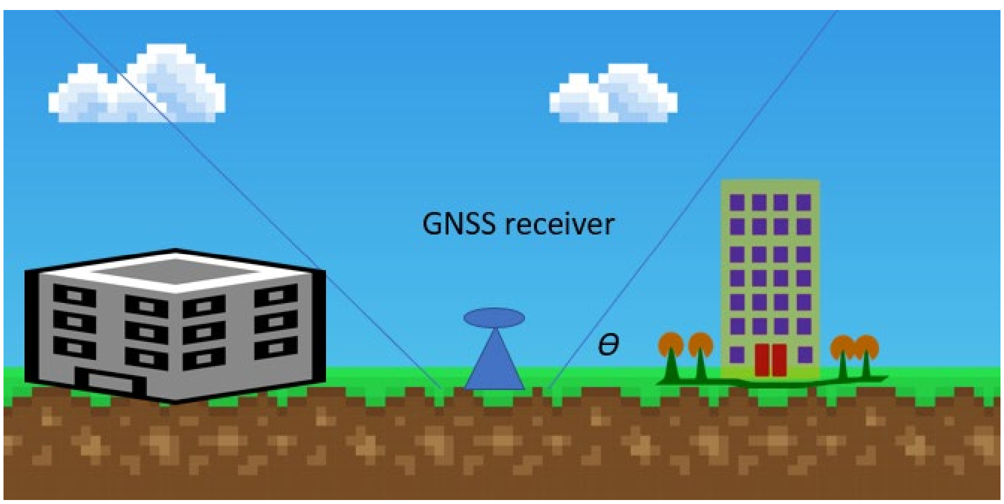

Figure IV.C.3 Graphic showing recommended GNSS location near tall objects.

8. Points shall not be placed near surfaces with abrupt changes in elevation such as curbs, buildings, or trees.
9. Points shall not be within 10 meters of tall features.

C.5.2 VVA points

As explained in the ASPRS Accuracy Standards, in vegetated areas where many lidar pulses are fully reflected before reaching the ground, a higher-density dataset tends to be more useful because more points will penetrate through vegetation to the ground. Additionally, more ground points will result in less interpolation between points and improved surface definition because more characteristics of the actual ground surface will be measured. Thus, the use of more ground points is more critical in variable or complex surfaces (such as mountainous terrain) where generalized interpolation between points would not accurately model all the changes in the surface. Therefore, while selecting VVA point locations, the following factors should be considered:

- If sunlight cannot directly reach the ground, then it is likely that the lidar will also not reach the ground. Hence, VVA points should not be placed under dense canopies or other vegetation where sunlight cannot penetrate.
- In vegetated areas where few lidar pulses will reach the ground, fewer points spaced farther apart will need to be used to interpolate elevation at the point. Therefore, VVA points should not be placed near surfaces with abrupt changes in elevation such as curbs, buildings, or trees.
- Points should not be within ten meters of tall features. As a general guideline, an inverted cone of θ = not more than 30-45 degrees at 1-meter distance from the point should be free of any obstruction.

C.6 Ground Survey Accuracy

The required accuracies of the GCPs, NVAs, and/or VVAs depend on the specified ASPRS accuracy class of the lidar product. According to the ASPRS Accuracy Standards, GCPs and NVA/VVA checkpoints should be at least twice as accurate as the lidar data's specified ASPRS vertical accuracy class.

C.7 Ground Survey Procedure

Surveyors should use Addendum II: Best Practices and Guidelines for Field Surveying of Ground Control Points and Checkpoints of the ASPRS Accuracy Standards to determine the best surveying method for establishing ground survey points. Depending on the project location, different control station configurations and densities, CORS stations, and Virtual Reference Stations (VRS) may be available; therefore, appropriate GNSS survey methods should be adopted to achieve the desired level of accuracy.

SECTION D: LIDAR DATA ACQUISITION

Airborne lidar data must be acquired in a manner that allows for adequate processing according to the requirements of the project. The following section will review best practices for airborne lidar acquisition.

D.1 Preflight

There are several preflight tasks that must be completed before the aircraft leaves the ground:

D.1.1 Flight Plan

The flight plan must be copied onto the flight management computer in the aircraft before takeoff. It may also be convenient to have a hard copy of the flight plan in the aircraft, since multiple screens are often used at the same time during airborne data collection.

D.1.2 Environmental Factors

Environmental factors, such as weather, smoke, standing water, snow cover, stream/river levels, and tides must be considered.

All lidar collections must be free of clouds, fog, and smoke below the aircraft. Clouds may be present above the flight altitude, but be careful not to collect data too close to the base of the cloud layer, as this can significantly increase recorded atmospheric clutter in the lidar data. Cloud shadows on the ground pose no issue for airborne lidar.

Airborne acquisition must occur during times free of precipitation, so as to not interfere with the laser ranging recordings. Additionally, standing water after a rain event will likely cause inaccurate laser recordings, and potentially even holes or drop-outs in the data. Similarly, snow cover may falsify the ground/feature elevations and should not be present during lidar collection unless agreed upon by the client. This topic must be addressed with the client before commencing with collection and should be noted in the contract.

Coastal projects will need to take into account tidal changes. As best practice, lidar acquisition for tidal areas should be performed at low tide to allow for the collection of the most topography. Likewise, stream and river levels should be within their banks.

D.1.3 Preflight Checklist

The remainder of tasks may be included in the preflight checklist. These may vary according to project needs and purpose, but a basic checklist should include the following:

- The sensor cover should be removed and the aircraft camera hole cover opened (if applicable).
- The collection day's PDOP, minimum number of satellites, and K-index (a.k.a. "KP index") plots should be reviewed for appropriate collection durations.
- The time of day should be appropriate to the lidar sensor. Some sensors perform best when flown at night.
- Lidar storage media should have enough free space for the planned collection timeframe.
- The lidar glass should be cleaned.
- If not using a GNSS post processing network reference station service (such as Applanix's PP-RTX or Inertial Explorer's TerraStar-NRT), the following should be reviewed:
    - GNSS base stations are set up (if applicable).
    - CORS stations (or other reference station network) are operational and recording at appropriate time intervals (usually ≤5 seconds).
    - An operational map is available in the aircraft to show how much of the geography falls within the appropriate radii of the ground reference stations. Consult manufacturer

recommendations for your Position and Orientation System (POS) for these maximum operational radius distances. They typically range between 50–100 km.
    - Coordinates of the ground reference stations are available in the aircraft for in-air GNSS initialization purposes.
- Planning and communication with ATC/TRACON regarding airspace flight notifications, permissions, or flight restrictions must be taken care of before takeoff, when and where applicable.

All lidar sensor manufacturers will have their own specific preflight checklists. It is strongly advised that flight operators adhere to these preflight checklists, as these typically go into more sensor details than what is listed in this document. Likewise, pilots should have their own aircraft preflight checklist.

D.2 Inflight

Once airborne, there are a few best practices that should be followed before data collection begins:

D.2.1 GNSS and IMU Initialization

If one or more GNSS ground reference stations are used, it is best practice to orbit around them for GNSS initialization. Follow the POS manufacturer’s recommendations for this procedure. This may not be needed if a GNSS post processing network reference station service is used with an appropriate aircraft antenna and POS hardware/software combination. The IMU must be initialized before and after collection. The standard procedure for this is to fly an S-curve pattern before lining up for the first line and after the last line is collected. Slight porpoising (up and down movement) will also assist the IMU initialization with respect to pitch.

D.2.2 Collection Approaches

There are varying approaches to collecting lidar flight lines. The following are best practice recommendations to yield the most accurate end products.

At least one crossline should be flown before beginning flight line collection, so that, should the mission be cut short for any reason (weather, sensor issues, etc.), the production lines have a crossline to be used for boresight and/or flight line adjustment. Generally speaking, each block of parallel flight lines should have at least one crossline, though two (one at either end) is better. It is best practice to collect the portions of the crosslines that overlap with the associated flight lines all in the same mission; this aids in the post-processing and will help create quality geometric strip alignment for the best potential relative and absolute lidar accuracy. Follow the system manufacturer’s recommendations for calibration flight patterns. Some third-party post processing software may also have recommendations or requirements for flight line geometry to yield accurately-calibrated data.

When possible, it is best to fly flight lines in alternating directions. Not only is this usually the most efficient flight pattern, but it also aids in the post processing calibration steps. Pairs of flight lines flown in the same direction is not cause for rejection, but this should not be the norm.

Areas at high risk for limited acquisition opportunities (such as higher-elevation zones that may get rain or snow sooner than lower elevation zones) should be prioritized. Areas that frequently experience clouds or fog should be flown when clear conditions are available. If leaf-off collection is preferred or required, consider the land cover of the project and prioritize collection of vegetated areas first (ideally during springtime). Lastly, locations with limited airspace should also be prioritized when that airspace is available.

D.2.3 Flight Navigation Tolerances

Acquisition tolerances will vary depending on the flight plan and the sensor used. Fixed FOV lidar sensors have different flight tolerances than dynamically-adjustable FOV systems. Most of this should be handled in the flight planning stage, but some sensors can make on-the-fly adjustments during collection to accommodate unexpected aircraft movement, while others may have very rigid operational tolerances. Always understand your specific sensor's limitations, as well as each flight plan's specific constraints. The tolerances to be aware of are:

- Maximum horizontal offset to planned flight line: affects sidelap
- Maximum vertical offset to planned flight line altitude: affects sidelap and point density
- Maximum heading deviation while online: affects sidelap
- Turbulence: affects sidelap and the consistency of point density
- Maximum bank angle during turns: affects GNSS satellite loss and 3D positional accuracy
- Maximum time on-line: affects IMU drift, thus affecting relative and absolute accuracy

It is important for the flight operator to watch the POS indicator lights to ensure quality data collection.

D.2.4 Typical Flight Tasks

Most airborne lidar systems include some form of real-time quality checking. If available, this should be used to look for gaps in coverage, watch for system issues, and monitor any other QC products or functions that it provides.

Flight logs should always be filled out for each mission. The information in these flight logs will likely vary by company or project, but the minimum attributes should be:

- Project name
- Date
- Mission
- Aircraft (including tail number)
- Sensor (including serial number)
- Crew
- A list of acquired flight lines according to line identifier
- Issues or observations worth noting

If any lines are not flown in their entirety, ensure that partial reflights of those lines overlap the original data to ensure full coverage. If applicable, patch lines should be flown as soon as possible to reduce temporal differences between acquired lidar data, as this can sometimes have an adverse effect on calibrating patch lines with the original flight lines.

D.3 Postflight

After each day of lidar acquisition, the flight operator must first make a copy of the data. This duplicated data is usually sent to the office and/or archived in some manner in case the original system drive fails or is otherwise damaged or lost. Each company will have their own procedures for moving acquired lidar and POS data to a data server. This can include external drives, data uploading or synchronization using an FTP or cloud service, or physically transporting the data. This step is critical for getting the data into the next phase for POS processing, data checking, and calibration. It is also important to free up space on the system drive. However, note that the data should only be cleared from the system drive after confirmation of data duplication has been received.

The flight operator will also need to update others on the progress of the day's data collection. This may include supervisors, colleagues, and/or the clients.

If base stations are required, batteries will need to be charged to prepare for the next day's work.

SECTION E: LIDAR DATA PROCESSING

E.1 Sensor Calibration

Accurate lidar point cloud data cannot be achieved without a well-calibrated lidar system. Airborne lidar systems are complex, and often need to be described by sensor-specific sensor models. In addition to lidar boresight angles and lever arms, there are many system-specific internal parameters that must also be calibrated. System manufacturers typically calibrate systems prior to delivery, a practice called factory calibration. Manufacturers will usually provide a calibration certificate, along with any calibration files that need to be uploaded to the system and/or used during lidar point cloud processing.

Calibration parameters may include range bias, scale factor, encoder scale factor and offset, various latency values, and various intensity-related parameters. For multi-channel systems, there may be additional angles and offsets describing the spatial relationships of the channels. Calibrating these parameters typically requires a combination of laboratory calibration and field calibration procedures. Field calibration typically involves collecting a special flight pattern (typically consisting of flight lines flown at multiple altitudes and special cross lines over a calibration field with control points). Sensor manufacturers typically include a recommended maintenance schedule describing how often the system should be sent in for a full factory recalibration—typically every two years. It is good practice for data providers to follow these recommendations. The recommended maintenance schedule does not apply to boresight angles. Boresight angles should be assessed on every mission and recalibrated as often as needed. Factory calibration, or at least the calibration of affected parameters, should also be repeated in the event of major changes or repairs to the system in between scheduled maintenance cycles.

Many internal calibration parameters are proprietary and specific to the sensor manufacturer. Some sensor manufacturers provide tools that expert data producers can use to refine some of these internal parameters between recommended maintenance cycles if needed. Additionally, there are a few software packages on the market that can model some of these calibration parameters using generic sensor models. Some of these software packages can work quite well in modeling errors, but it is not always possible to establish an exact relationship between generic calibration parameters and the sensor-specific sensor model parameters. Therefore, the calibrated parameters cannot be easily used to process future missions.

E.2 GNSS/IMU Processing

The first step in the lidar data processing workflow is to process the acquired airborne GNSS/IMU data. This processing is accomplished via hardware-specific software, and there are a number of GNSS/IMU processing techniques in use. Some use one or multiple base stations or CORS stations (or other continuously operating reference station network) for differential GNSS processing, while others (such as the Trimble/Applanix PP-RTX or the Hexagon/Novatel TerraStar) require a subscription to a GNSS correction service to provide correction parameters for processing. Precise Point Positioning (PPP) can be another option, depending on the required accuracy of the final product. Processing methods can also be categorized in terms of loosely or tightly coupled processing. The data provider should select an appropriate processing technique that can support the desired lidar accuracy.

Many details need to be considered during GNSS/IMU processing. The data provider should ensure that the correct GNSS lever arms are applied during processing and IMU mounting angles are properly defined. Most GNSS/IMU processing software allows for the estimation of the GNSS lever arms, and it is good practice to double check or in some cases refine the known GNSS lever arms before they are used for processing the airborne trajectory.

If one or more GNSS base stations are used, their coordinates must be determined with an accuracy that can support the required lidar accuracy class. In addition, the antenna height and antenna type should be provided. Special care should be taken to ensure that the datum of the base station coordinates is properly defined in the GNSS/IMU processing software; if the datum is not properly defined in the POS processing, it can introduce offsets into the processed lidar.

The GNSS/IMU solution may be processed in reference to either the lidar reference point or to the IMU reference point, depending on company practices. If the GNSS/IMU solution is in reference to the IMU reference point, then the internal laser lever arms provided by the lidar system manufacturer should be applied in the lidar point cloud processing software. If the lidar system is mounted on a Gyro Stabilized Mount (GSM) to compensate for the roll, pitch, and yaw/heading movements of the aircraft during flight, the GNSS/IMU solution should be processed in reference to the GSM reference point.

Depending on the software used, there are various plots that can be reviewed to ensure accuracy for the GNSS/IMU solution. Some of the plots to review include:

- Solution status plots: Includes the number of satellites, Position Dilution of Precision (PDOP), and the processing mode (whether the GNSS ambiguity was fixed or float). Float ambiguities should be avoided. If single baseline processing was performed, then check to make sure maximum baseline length is within project requirements, if applicable.
- Smoothed best estimate of trajectory plots: Review start/end times to ensure the project is complete. Verify that the altitude plot appears as expected for the mission, and review roll, pitch, and true heading plots to verify the IMU orientation used in post processing.
- Forward/reverse separation plots: Provide a good indication of the accuracy of the solution.

- The Kalman Filter's estimate for the IMU errors: Should be reviewed to ensure IMU performance is within manufacturer recommended limits.
- Gimbal data plots: Review if a gimbal mount was used.
- Ensure the smoothed performance metrics (North, East, Down (N, E, D); and roll, pitch, yaw/heading RMS values) are within accepted limits for the project.

The accuracy of the final GNSS/IMU solution should be better than the required lidar accuracy.

Some GNSS/IMU software packages provide an option to batch process multiple missions to increase efficiency. This can be dangerous if the solution for each mission is not carefully checked afterwards.

The GNSS/IMU processing is performed in WGS84/ITRF. If the lidar project requires a different delivery datum (such as NAD83(2011), which is typical for projects in the United States) then the smoothed best estimate of trajectory in the required datum can be exported from the GNSS/IMU software. Alternatively, the datum transformation can be performed by the lidar processing software when exporting the lidar point cloud.

E.3 Lidar Point Cloud Processing

To derive the lidar point cloud, the raw laser data must be georeferenced using the processed GNSS/IMU trajectory. The processing is typically performed using hardware-specific software. In the event that waveform data was collected, then an extraction algorithm must first be run to extract discrete returns from the full waveform data. During lidar point cloud processing, the measured laser ranges and scan angles are combined with the GNSS/IMU trajectory to compute the 3D position of each return in the point cloud.

During processing, it is important to ensure that proper system calibration parameters are used, including sensor specific calibration parameters (usually provided by the sensor manufacturer). Additionally, it is good practice to keep track of changes in the system’s boresight angles and to ensure they are kept up-to-date. If the GNSS/IMU solution was referenced to the IMU or the gimbal mount, then the laser lever arms (also typically provided by the sensor manufacturer) need to be defined in the point cloud processing software as well. Atmospheric correction parameters, including flying altitude, temperature, atmospheric pressure, and humidity, must also be provided to correct for any range errors due to atmospheric conditions. Depending on the software, the GNSS/IMU trajectory may need to be in a certain datum, or its datum will need to be specified. The desired point cloud processing datum, coordinate system, and units will also need to be defined. Special care should be taken when setting up all the processing parameters, as any mistake can introduce systematic errors into the lidar point cloud.

It is good practice to record the calibration parameters used to process each flight line, to ensure traceability in the event one or more flight lines need to be reprocessed at a later point. Most lidar point cloud processing software automatically tracks this information, or at least provides an option to save it in the settings.

Lidar point cloud files are typically formatted as LAS or LAZ (a losslessly compressed form of LAS) files; file type is usually specified in the project requirements.

Many lidar point cloud processing software packages offer some visualization options, to allow for quick verification of coverage and to provide a way to make an initial check of flight line overlap mismatches.

E.4 Initial QC of Point Cloud

Before any further processing of the lidar point cloud, an initial QC should be performed to check for the following:

- Check coverage against project boundaries.
- Make sure planned sidelap was achieved and that there are no gaps between flight lines. For projects that were planned to achieve point density requirements with double coverage, the achieved sidelap should be ≥ 50%.
- Check for environmental factors such as clouds that may affect the point cloud.
- Check for unusual dropouts (missing returns), especially on low-reflectivity targets.
- Make sure all planned cross ties and calibration lines were collected.
- Check that point density and distribution meet requirements**.**
- Check for any unexpected anomalies in the data that could indicate system malfunction.
- Check for range gate issues or Multiple Time Around (MTA) processing errors.

Flight lines or segments of flight lines which do not meet project requirements should be reflown. It is best practice to establish a tracking methodology to keep track of accepted/rejected flight lines and reflights.

E.5 Boresighting and Flight line Adjustment

Even if the lidar system is well-calibrated, relative error in flight line overlap areas is often significant, especially in datasets that were collected at higher flying altitudes. These relative mismatches are often caused by small changes in boresight angles and/or errors in the GNSS/IMU solution. Depending on the relative accuracy requirements for the dataset, boresight calibration and/or flight line adjustment may be necessary to minimize these relative line-to-line mismatches.

E.5.1 Treatment of Blunders

Prior to performing any flight line adjustments on the dataset, it is good practice to check the point cloud for absolute accuracy compared to GCPs, as well as relative line-to-line mismatches. If significantly larger absolute errors or line-to-line mismatches are present in the dataset than what can be reasonably expected, then the source of the error should be investigated beforehand. Examples of such blunders include: accidentally processing the data using the wrong calibration, errors in the lever arms used for GNSS/IMU processing, errors in the base station coordinates or wrong antenna height used for GNSS/IMU processing, processing the dataset in the wrong datum, etc. System malfunctions can also result in serious anomalies in the point cloud.

E.5.2 Boresighting

Boresight is the small angular misalignment between the laser and IMU reference frames. It consists of roll, pitch, and yaw/heading angles. Small boresight angle changes can be caused by various factors, such as hard landings or changes in temperature. It is not unusual to see small boresight angle changes from mission to mission. If the boresight angles shift enough to significantly affect the accuracy of the dataset, then the boresight angles should be adjusted. Boresight adjustment requires certain flight line geometry, such as opposing flight lines and cross ties, to be able to separate the boresight angles. For multi-channel sensors, any relative channel-to-channel mismatch will negatively affect the surface quality of the point cloud. Therefore, any channel mismatches should be identified and corrected.

E.5.3 Flight line Adjustment

After boresight errors are removed, additional flight line adjustments may need to be applied to minimize any line-to-line relative errors still remaining in the data. Flight line adjustments should be based on the flight line trajectories when computing roll, pitch, and yaw/heading adjustments, as well as small X, Y, Z offsets per flight line if necessary. In some cases, it may be necessary to allow additional time-dependent drift adjustments to correct for drift along the flight line. The parameters that can be adjusted depend on the available flight line geometry. When available, cross ties should be included in the adjustments in order to strengthen the geometry. While vegetation and water bodies should be excluded from the flight line matching process, the computed corrections should be applied to all points in the point cloud.

E.5.4 Use of GCPs and Checkpoints

It is good practice to include the GCPs in the flight line adjustment. Alternatively, a small vertical adjustment may be applied to the data to fit the point cloud to the GCPs after flight line adjustment; this is commonly known as debiasing.

After flight line adjustment, prior to beginning the ground filtering, checkpoints should be used to determine and verify the initial vertical accuracy of the unclassified data. Checkpoints cannot be included in the flight line adjustment; they have to be kept out of the adjustment for independent verification of the achieved accuracy.

E.5.5 Types of Adjustments to Avoid

Constrained adjustments of the point cloud data after the application of proper system calibration and boresights values can mitigate GNSS/IMU solution error, but it is important to avoid altering the data in a manner that excessively overrides inputs from the system calibration, system boresight, and GNSS/IMU solution in order to fit the data to the GCPs and/or improve internal precision. This is known as rubbersheeting, and it is a type of data manipulation that will compromise the integrity of the lidar point cloud data. Rubbersheeting often manifests in improved internal data precision at the cost of absolute accuracy, and it can generally be identified by poor correlation to ancillary ground truth data.

The point cloud data should be adjusted in a way that maintains the integrity of each flight line. When using tiled point clouds in the adjustment process, lidar tiles should not be adjusted independently of surrounding tiles, as this could create discontinuities at tile edges.

E.5.6 Handling Multiple Blocks

When the lidar dataset is divided into multiple blocks for flight line adjustment, either due to incremental delivery requirements or to break large datasets into manageable sections, it is important to consider the relative errors between overlapping parts of the newly adjusted block and previously adjusted blocks. One approach is to use overlapping segments of previously adjusted blocks as reference data when adjusting the new block.

E.6 Testing and Reporting the Achieved Absolute and Relative Accuracy

Before classifying and developing any products from the lidar point cloud, the absolute and relative accuracy of the point cloud should be evaluated to ensure that the required accuracy was achieved. Absolute accuracy should be tested and reported in the final deliverable datum and projection.

E.6.1 Testing Absolute Vertical Accuracy

Both Non-Vegetated Vertical Accuracy (NVA) and Vegetated Vertical Accuracy (VVA) of the point cloud data should be assessed.

NVA for the point cloud data should be assessed by comparing checkpoints surveyed for NVA assessment to a Triangulated Irregular Network (TIN) constructed from ground-classified lidar points in those areas. The fit of the data to the NVA checkpoints is expressed as $RMSE_Z$. When computing the Vertical Positional Accuracy ($RMSE_V$) based on the NVA points, the accuracy of the checkpoint survey must also be factored in (as described in section 7.12.4 of these Standards). The computed Vertical Positional Accuracy should be equal to or less than the desired vertical accuracy class of the lidar data.

VVA for the point cloud data should be assessed by comparing checkpoints surveyed for VVA assessment to the TIN constructed from ground-classified lidar points in those areas. According to the ASPRS Accuracy Standards, the fit to the VVA checkpoints should be computed as $RMSE_Z$, with care taken to evaluate skew and kurtosis. Skewed results may occur in vegetated areas due to the low density of the lidar point cloud and the degraded quality of GPS surveying under trees. When computing the Vertical Positional Accuracy ($RMSE_V$) based on the VVA points, the accuracy of the checkpoint survey must also be factored in. VVA should be evaluated and reported, but only the reported NVA should be used to make a pass/fail decision of the dataset, regardless of whether the VVA meets the specifications.

If the project requires a digital elevation model (DEM) to be derived and delivered, the NVA and VVA for the DEM must be calculated and reported. NVA and VVA of the DEM can be assessed by comparing checkpoints to the final bare-earth surface. Appendix C of the ASPRS Accuracy Standards describes how to determine the vertical position of the checkpoints in the lidar point cloud data and DEM. NVA of the DEM can be reported as $RMSE_V$, similarly to the NVA reporting of the point cloud data.

E.6.2 Testing Absolute Horizontal Accuracy

The data producer and user should determine whether horizontal accuracy testing is required for the project and what the desired horizontal accuracy should be. Currently, there is no one widely accepted horizontal accuracy testing method for lidar data. Some approaches use painted road markings and/or specially designed targets to test lidar horizontal accuracy.

If horizontal testing is required, then the achieved horizontal accuracy should be calculated by comparing the surveyed positions of the horizontal checkpoints with their positions in the lidar data. To report the horizontal accuracy, $RMSE_X$, $RMSE_Y$, and $\mathrm{RMSE_{H_1}}$ should be calculated from the checkpoint residuals, as described in Section 7.12.1 of these Standards.

When computing the Horizontal Positional Accuracy, the accuracy of the checkpoint survey must also be factored in, as described in section 7.12.3 of these Standards. The computed Horizontal Positional Accuracy ($RMSE_H$) should be equal to or less than the desired horizontal accuracy class of the lidar data.

If horizontal checkpoints are not required for the project, then the estimated horizontal accuracy of the lidar data can be calculated based on the flying altitude and the GNSS/IMU accuracy using the formula described in Section 7.6 of the ASPRS Accuracy Standards.

E.6.3 Handling Blunder Checkpoints

When a checkpoint residual in X, Y, or Z exceeds three times the required RMSE threshold, the checkpoint should be considered a blunder. Blunder points should not be discarded without proper investigation to find out the source of the issue. Some potential problems include:

- An object (such as a vehicle) was obscuring the checkpoint during lidar data collection
- The checkpoint was not surveyed in a relatively flat area suitable for lidar checkpoint
- The surveyed coordinates of the checkpoint have a blunder

If the source of the error can be identified, then, if possible, it should be corrected, and the corrected checkpoint should be included in the final accuracy reporting. If the error cannot be corrected, the blunder checkpoint can be excluded from the final accuracy calculation and reporting if proper explanation is provided. Appendix C of the ASPRS Accuracy Standards lists criteria that must be met for a checkpoint to be excluded from the accuracy assessment calculations.

E.6.4 Testing Data Internal Precision

Data internal precision refers to the internal geometric quality of a lidar dataset without regard to surveyed GCPs. The assessment includes two aspects of data quality: within-swath accuracy (smooth surface precision), and swath-to-swath accuracy.

For calculating smooth surface precision and swath-to-swath relative accuracy, the methods described in section 7.15 of this document should be followed. The acceptable limits for each ASPRS accuracy class are listed in Table 7.2 of the ASPRS Accuracy Standards.

E.6.5 Reporting of the Achieved Accuracy

Depending on whether checkpoints were available for accuracy testing, and whether the number of checkpoints used for accuracy testing meets the minimum required number of checkpoints for the project size as described in the ASPRS Accuracy Standards, the Horizontal, Vertical, and Three-Dimensional Positional Accuracy of the lidar dataset shall be documented in the metadata in one of the manners described in Section 7.16 of this document.

ADDENDUM V: BEST PRACTICES AND GUIDELINES FOR MAPPING WITH UNMANNED AERIAL SYSTEMS (UAS)

Contents

FIGURES

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PURPOSE

This Addendum contains general best practices and guidelines for UAS-based mapping techniques. These best practices have been divided into two sections: Section A focuses on UAS-based photogrammetric methods, while Section B focuses on UAS-based lidar.

SECTION A: BEST PRACTICES AND GUIDELINES FOR MAPPING FROM UAS-BASED PHOTOGRAMMETRY

A.1 Objectives

Section A of this Addendum is intended to provide best practices and guidelines for selected aspects of photogrammetric surveying using an Unmanned Aerial System (UAS). It is designed with three objectives in mind:

1. To provide producers with appropriate common guidelines and recommendations for acquiring accurate data via topographic airborne photogrammetry UAS systems, without regard to any specific sensor or manufacturer.
2. To assist companies and agencies in establishing standards for their organizations.
3. To help increase efficiency and accuracy of data collection.

These guidelines are designed to support the users of this Addendum in producing accurate and precise mapping products with integrity and professional responsibility. These guidelines are not intended to be a comprehensive documentation of the UAS photogrammetry process.

A.2 Unmanned Aerial System Platform Type

There are three common unmanned aerial system (UAS) aircraft types: multi-rotary, fixed-wing, and hybrid. A hybrid aircraft is a combination of the rotary-wing and fixed-wing aircraft types, a configuration which may allow a fixed-wing aircraft perform vertical take-off and landing (VTOL). Some advantages and disadvantages of each aircraft type are highlighted in this section.

A.2.1 Fixed-Wing Platform

Fixed-wing aircraft (except those with VTOL capabilities) typically require adequate launch and landing zones, similar to piloted aircraft.

During flight, fixed-wing aircraft may take wide turns, which limits maneuverability. It is important to know the turn radius of the aircraft, especially for projects with areas adjacent to roads or featuring vertical boundaries such as buildings, trees, or airspace restrictions. These factors can pose logistical challenges (both physical and regulatory) when making turns. Additionally, small UAS fixed-wing platforms may experience turbulence, and may struggle to maintain consistent photo overlap. This often leads to higher degrees of angular exterior orientations compared to data collected from larger and more stable aircraft. Additionally, small fixed-wing aircraft are particularly susceptible to crab angle effects when flying into the wind.

Advancements in camera-mounting techniques, such as gimbal mounts, have mitigated some of these angular orientation problems. Processing software based on Structure-from-Motion (SfM) algorithms, explained in more detail in Section A.9: Processing, can handle large fluctuations in angular orientation, provided that the user planned for such eventualities and/or is familiar with the behavior of the aircraft. Additionally, fixed-wing aircraft are capable of gliding, lending them excellent endurance and good overall efficiency when mapping large project areas.

A.2.2 Rotary-Wing Platform

Rotary-wing aircraft use propellers that keep them in the air, giving them the ability to hover and fly in reverse. These aircraft do not require as much airspace to make turns for sequential flight lines compared to fixed-wing aircraft. Although not resistant to the negative impacts of turbulence, rotary-wing aircraft are generally more stable than fixed-wing aircraft, granting them more consistent endlap (or forward lap) and sidelap, and they can be flown slower and at lower altitudes. Additionally, rotary-wing aircraft are ideal for terrain-following, as their excellent maneuverability allows them to maintain a consistent elevation between the camera and the terrain with greater ease than fixed-wing craft.

However, rotary-wing aircraft are much less power efficient, and may require several battery replacements when collecting data over large sites and/or carrying heavy payloads.

A.2.3 Hybrid Platform

Hybrid aircraft have the advantage of a small takeoff/landing footprint while maintaining the flight efficiency of fixed-wing platforms. However, once airborne, hybrid aircraft are subject to the same limitations as fixed-wing platforms.

A.3 Camera System

This Addendum focuses on Red, Green, Blue (RGB) sensor cameras, and broadly categorizes cameras into one of two types: metric and non-metric. Both types of cameras are integrated within UAS.

A.3.1 Metric Cameras

Metric cameras are often specifically constructed for precise photogrammetric mapping or metrology applications. Physical characteristics of metric cameras may include optimized unit size for external mounting, specialized interchangeable lens interfaces, precise lens constructions, advanced sensor technology, medium-to-large format sensors, enhanced color fidelity, optimal frames-per-second rates, and optimized mid-exposure pulse (MEP) signalizations. Metric cameras may have mostly rigid internal components, which are constructed to have a high degree of relative stability and precision. Thus, interior orientation calibration parameters (including lens distortion, principal distance, and principal point) can be modeled with repeatable values.

In addition to the quality of the camera construction, a proper metric camera will have an MEP that is synchronized to both the Global Navigation Satellite System (GNSS) antenna receiver and the Inertial Measurement Unit (IMU), enabling optimal integration into the UAS Inertial Navigation System (INS). Metric cameras come in various grades, and UAS aircraft manufacturers may include a metric camera integrated into the craft that has been optimized specifically for precise photogrammetric mapping. Depending on the grade of the UAS, this system may or may not include IMU integration.

A.3.2 Professional Consumer and Consumer-Grade Cameras

Consumer-grade cameras are commonly incorporated into UAS systems for precise photogrammetric and lidar mapping. Only mirrorless digital cameras will be considered here; Digital Single-Lens Reflex (DSLR) cameras will be excluded, as the mechanical nature of the shutter mechanism may cause movement during exposure and the cameras themselves tend to be cumbersome.

Consumer-grade cameras may not have the same degree of precision, internal component stability, or integrative interface for GNSS synchronization that can be expected in metric cameras. However, some UAS manufacturers are starting to integrate professional consumer cameras into their platforms, which may lead to increased technical compatibility between UAS and consumer-grade camera models.

Many professional consumer-grade cameras have interchangeable lenses, allowing the user to upgrade this crucial hardware component to a more stable, higher-quality lens. Additionally, it is possible to pre-calibrate consumer-grade cameras to determine true focal length, lens distortion, and principal point parameters; but, due to slight relative internal instability compared to metric cameras, these values may not be as repeatable. Thus, a consumer-grade camera may require calibration more frequently, and data processing may require the use of SfM combined with camera self-calibration algorithms in order to sufficiently compensate.

Lower-grade consumer cameras may not be suitable for photogrammetric and lidar mapping, as these tend to have more moving parts (such as multi-part lenses) to accommodate a wide variety of scene capture scenarios. In action, they may perform undesired automatic adjustments, which can be detrimental to data collection. These types of non-metric cameras have been extensively researched within early modes of UAS. If used, greater care and consideration must be taken to achieve optimal accuracy, including accurate determination of the mid-exposure pulse within an onboard positioning system.

A.3.3 Common Shutter Types: Rolling and Global Shutters

The two most common shutters that appear in UAS photogrammetry are rolling and global shutters. Both shutters can be mechanical, electrical, or a combination of the two.

A.3.3.1 Rolling Shutter

A rolling shutter operates using a line-scanning motion during image capture, where pixel readouts occur at different times across the sensor array. The rolling shutter effect may present itself as a distortion in the image caused by the fast motion of the UAS during capture. Depending on the speed and direction of the aircraft relative to the scan line direction, static objects may be shortened, elongated, or skewed. Abrupt movements of the aircraft during exposure may cause distortions that are difficult to model. Image capture may also be affected by flashing lights during capture.

Acceptable photogrammetric data has been collected using these shutters, and, provided the imagery processing software offers rolling shutter compensation features (either inherently or explicitly), these shutters are capable of producing optimal results. It should be noted that, although the rolling shutter has advantages in efficiency, the mechanism is generally less desirable for use in UAS photogrammetry, for reasons outside of the scope of this Addendum.

A.3.3.2 Global Shutter

In a global shutter, pixel readouts occur across the sensor simultaneously. In other words, the shutter captures the entire frame in a single exposure, with all pixels being exposed to light at the same time. The calibration and distortion correction of imagery captured by the global shutter mechanism are more straightforward and more easily modeled, though these shutters tend to have more image noise and have less efficient frame rates in proportion to sensor resolution. Leaf shutters, a type of global shutter, have multiple blades which open radially toward the edge of the frame, causing the center of the photograph to be exposed for slightly longer than the edges.

A.3.4 Sensor Types

There are two sensor types mentioned here: the Complementary Metal Oxide Semiconductor (CMOS) array and the Charge Coupled Device (CCD) array. The CMOS array is the most common sensor found in digital cameras, and it may be coupled with a mechanical, electrical, or combination/hybrid shutter. The CMOS sensor was originally coupled with a rolling shutter mechanism, but today it may feature electronic global shutters, mechanical leaf shutters, or curtain shutters.

A.3.5 Focal Length

The camera focal length may be fixed (prime) or adjustable. In metric camera lenses, fixed focal lengths are preferred over adjustable ones, as these tend to produce superior image qualities and provide added stability to the interior orientation of the camera.

A.3.6 Exposure Modes

Most cameras have several exposure modes: auto, manual, shutter priority, and aperture priority. Auto mode allows the camera complete autonomy when determining shutter speed, aperture, and film speed (ISO) values. While this mode may work in many cases, it is the least-preferred setting when collecting data for photogrammetric and lidar mapping, as the camera may make undesirable adjustments, such as setting an extremely low shutter speed and thereby causing the data to contain excessive motion blur. Therefore, the user should understand the photographic balance between aperture, shutter speed, and ISO (i.e., the sensor sensitivity to light) in cases where adjustments may need to be made.

Manual mode requires the user to set (and typically lock) the aperture, shutter speed, and ISO. Shutter priority mode requires the user to set (and typically lock) only the shutter speed, while allowing the camera to automatically determine aperture and ISO values. Aperture priority mode requires the user to set (and typically lock) the aperture value, while allowing the camera to automatically determine shutter speed and ISO. The user should be familiar with the functionalities and limitations of each to achieve the best results.

A.3.7 Metering Modes

Metering modes allow the user to define which part of the frame the camera uses to determine exposure. Spot metering and matrix metering are two common metering modes, though they may go by other names depending on manufacturer. Spot metering allows the user to define a specific point of the frame to evaluate exposure, while matrix metering allows the user to set a specific area of the frame for the camera to evaluate exposure. Matrix metering is generally preferred, as it provides the camera with a “more average” value of the entire scene rather than a singular point.

A.3.8 Camera-Aircraft Mounting

Cameras may be mounted onto the aircraft using either a gimbaled mount or a fixed mount. Both fixed- and gimbal-mounted systems offer vibration dampening features. With a gimbal system, the camera may be set to any desired orientation relative to the flight direction. When using a gimbal system, it is important to note whether the photo orientation remains constant. Additionally, if the system inertial measurement unit (IMU) provides the exterior image orientation angles and the data producer uses these exterior orientation values for data processing and products generation, the metadata should define the source of these exterior orientation values.

In a fixed mounting system, the image orientation relative to the flight direction is constant and exterior orientation angles are closely tied to the movement of the aircraft, making any additional computations much simpler. Reliable and accurate photogrammetric results have been produced using both mounting systems, due to advances in computer vision algorithms utilized in UAS imagery processing software.

A.3.9 Other Camera and Sensor Types

Line cameras are unique in their method of data collection, and care must be taken when handling the resulting data. Other camera sensor types that are outside of the scope of this Addendum include thermal, multispectral, and hyperspectral sensors. These sensors and their resulting outputs are typically not in the domain of survey-grade mapping, but there are rare cases where they may be used, such as in thermographic mapping applications, which make use of thermal sensors, or in land cover classifications, which make use of multispectral and hyper-spectral sensors.

A.4 Onboard Positioning

In order to use an image to produce any mapping product, the image’s exterior orientation parameters must be known. Image exterior orientation consists of six parameters that describe the image position and orientation in 3D space, with respect to a local mapping frame of reference (or datum). These six parameters consist of three positional components (X, Y, Z) and three orientation angles (ω, ϕ, κ).

Today, virtually all mapping systems use onboard positioning in order to determine the positional components of the exterior orientation parameters via some form of Global Navigation Satellite System (GNSS, which includes multiple satellite constellations including GPS, GLONASS, Galileo, and BeiDou) receiving capabilities. GNSS receiving capabilities may be dual-frequency (L1/L2), multi-frequency (L1/L2 and L5), or single-frequency (L1) signals. Dual-frequency and multi-frequency signals are often called "survey grade," while single-frequency signals are mainly used for flight navigation and waypoint following, as the positions captured by single-frequency signals tend to be too inaccurate to be useful in photogrammetric computations.

Although ground control is highly recommended, exterior orientation parameters may also be measured, partially measured, or computed. These methods are described below:

Measured: The Direct Georeferencing (DG) method uses GNSS- and IMU-integrated systems to determine the six orientation parameters. Aerial triangulation is optional, and depends on IMU sensor quality, GNSS processing method, and map accuracy requirements. This method allows the user to compute a point location in a photograph instantly, and is useful in real-time response applications, since it does not require ground control points (GCPs). One GCP could be used to calibrate the local datum, if needed.

Partially-Measured: This method uses GNSS to measure the three positional components of image exterior orientation at the moment of image exposure, and then uses those values to compute the three remaining angular components for aerial triangulation. It may incorporate either Post-Processed Kinematic (PPK) or Real-Time Kinematic (RTK) positioning:

- PPK is the most accurate AT method and has the least running costs, since it requires minimal or no GCPs. PPK implies that the multi-frequency GNSS data acquired during UAS aerial image acquisition is fully processed using dedicated GNSS post-processing software incorporated with a dedicated base station or some other augmentation method or service. When GNSS PPK is used properly, it frequently reaches accuracies of 1–2 cm (horizontal) and 2–4 cm (vertical), as evidenced from many mapping projects in the past decade from around the world.
- RTK may be used in an environment where corrections are broadcast by a nearby multi-frequency base station and received by the UAS in real-time. If set up properly, this may allow the RTK to achieve typical accuracies in the 5–10 cm range, a value that is constantly improving as satellite constellations expand.

Computed: Aerial triangulation is the most traditional method in photogrammetry. It depends on the presence of a high number of well-distributed GCPs to accurately compute the exterior orientation parameters. It has been used since the 1980s and is a robust method with the fewest minimal initial costs for any UAS mission, since it does not depend on the presence of onboard measuring sensors such as GNSS receivers or IMUs. However, it has the highest running cost per project because the GCPs it depends upon must be established and surveyed for each project.

A.4.1 The Role of Ground Control Points

This Addendum will focus on a combination of partially-measured exterior orientation parameters and partially-computed parameters through AT methods and will assume that: 1) the aircraft is equipped with a dual-frequency L1/L2 GNSS antenna, and 2) that GCPs and checkpoints have been surveyed. Additionally, with the advancement in computer vision algorithms such as Structure-from-Motion (SfM), this Addendum will assume that the three positional exterior orientation values (i.e., X, Y, Z) are sufficient and do not require additional orientation angles measured via IMU.

While dual-frequency onboard positioning via RTK or PPK greatly mitigates the need to rely upon ground control, it is still highly recommended that GCPs be surveyed for use in the bundle block adjustment of the photogrammetric model as well as the accuracy assessment, as per the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2.0 (2024), referred to here as the ASPRS Standards. Onboard positioning can help optimize the adjustments to the data, but surveyed GCPs should take priority, as airborne RTK and PPK solutions have neither the necessary quantity of epoch readings nor the appropriate levels of geoid compensation to produce sufficiently accurate image centers, especially when compared to GCPs measured with multiple epoch readings using survey-grade equipment, which tend to produce more accurate positional values.

A.4.2 Mid-Exposure Pulse

The mid-exposure pulse (MEP) is the synchronization between the moment of image exposure and the GNSS signal reception. The MEP is typically resolved by the manufacturer or system integrator, but the MEP may not be sufficiently synchronized if the GNSS receiver was integrated into the aircraft after-market, or if the system has had one or more of its original sensors replaced with another sensor of a different type or manufacturer. Evidence of an unresolved MEP may appear as systematic horizontal shift or bias in the exposure station's residuals after adjusting for flight direction.

A.4.3 Cycle Slips

Cycle slips are the receiver's loss of lock on the GNSS signal. They can be caused by vertical obstructions, power disruptions, low signal-to-noise ratio, or severe ionosphere conditions. Larger, crewed aircraft flights are generally designed with a limiting banking angle, to support maintaining GNSS phase lock.

A.4.4 Inertial Measurement Unit

In the context of photogrammetric mapping with UAS, the Inertial Measurement Unit (IMU) typically used onboard the UAS is not accurate and should only be used for aircraft navigation purposes. Exterior orientation angles measured by an accurate IMU or determined through AT are necessary for photogrammetric processing. The IMU measures roll, pitch, and yaw/heading, which are transformed into ω, ϕ, κ in post-processing. These angles provide initial and/or accurate camera orientation values (depending on the IMU quality), which can be useful when mapping corridors or remote areas where it is not possible to establish ground control.

The decision to use an IMU as an integral part of a system depends on a number of factors, including technical, operational, budgetary, and logistical parameters. In order to achieve sufficient mapping accuracy levels, it is essential that the GNSS and IMU measurements be properly weighted and that a few ground control points be used in the AT processing.

A.5 Data Acquisition Considerations

A.5.1 Safety

Safety is essential to any UAS mission, and must be considered before, during, and after flight. It is important to be aware of one's surroundings and to maintain the safety of both the public and the project personnel. Reconnaissance should be performed on the project area to take note of any potential hazards, such as tall objects that may become obstructions during flight, as these may ultimately dictate factors such as minimum flying height. In certain circumstances, a manned aircraft may need to be used instead of a UAS.

A.5.2 Weather and Seasonal Considerations

Additionally, weather conditions during flight may affect aircraft attitude and heading and may negatively impact image quality. Image matching (or image stitching) requires identifying thousands of well-defined pixel features called key points, and then locating these same features in photos that overlap the area, which are called tie points. However, this process may be limited or uncertain in areas with dense foliage or homogenous textures (such as new asphalt with no paint or bodies of water). Additionally, low image brightness may result in unreliable key point matching in SfM solutions, as long shadows can potentially overwhelm the scene of capture by creating large, empty patches in the data.

In areas with less dense foliage, image matching becomes more reliable during fall or winter (also known as leaf-off conditions). External weather conditions such as wind, haze, high humidity, or wildfire smoke may yield low quality imagery or prevent safe flight operation altogether.

A.5.3 Ground Sample Distance

In digital imagery, ground sample distance (GSD) is the distance measured on the ground from pixel-center to subsequent pixel-center (assuming square pixels). The GSD depends on the flying height, focal length, and pixel size of the sensor. Because the GSD is effectively the spatial resolution of the raw digital image, it directly affects the achievable accuracy in the photogrammetric product. Please note that the GSD may change value after pixel resampling in the final orthophoto (or orthomosaic) product.

The minimum acceptable GSD is typically determined based on the project's desired level of accuracy. Planning software may provide automatic flight parameters to meet project specifications regarding GSD. Refer elsewhere in these Standards for more information regarding GSD.

A.5.4 Image Overlap

Overlap is a critical aspect of photogrammetric data collection. Forward lap is defined as overlap between subsequent images along the flight direction. Sidelap is defined as overlap between adjacent flight lines, or strips. Forward lap and sidelap are expressed in percentage of image area.

In traditional photogrammetry, the area of overlap between photographs allows for the creation of a stereo model and enables the computation of 3D points. The certainty of computed points depends largely on the geometry of the intersecting light rays that emanate from the point on the object to the sensor arrays of consecutive exposure stations in the air. In traditional photogrammetry, values for forward lap may fall around approximately 60%, and values for sidelap may fall around approximately 30%. These values may be higher (i.e. 80% and 60% for forward and sidelap) in UAS photogrammetry.

UAS photogrammetry uses Structure-from-Motion (SfM) algorithms to determine camera orientation at the instant of exposure. In SfM, camera orientation is computed via the intersection of multiple rays to common points on overlapping images. This process does not necessarily require a priori camera positions or angular orientation values. Higher forward lap and sidelap are recommended in UAS photogrammetry compared to traditional photogrammetry and may go up to 80% in some cases. SfM algorithms benefit from high overlap values since increasing the number of intersecting rays strengthens the geometric solution of object space coordinates of points, and since higher overlap helps with automatic image matching by reducing perspective changes in the features imaged in successive images. These higher overlap values are especially necessary to ensure coverage in areas with deep canyons or tall buildings. By using the ray redundancy created by these high overlap values, digital imagery processing software for UAS photogrammetry can even compensate for what may otherwise be poor geometry in high-altitude stereophotogrammetry. Users should be aware that planned overlap percentages will vary from the actual resulting values due to external factors.

Forward lap and sidelap consistency must be weighed against the undulating nature of the project site topography, the type of aircraft, weather conditions, and other factors that may cause deviation of overlap from planned parameters. Without sufficient forward lap, factors such as aircraft tilt, varying topography, and varying aircraft altitude may result in gaps in the data created by loss of coverage. Fixed-wing crab angles may reduce forward lap. Depending on the platform, certain flight planning software packages may offer a terrain-following feature, which allows the craft to receive elevation information prior to flight, usually from a surface or elevation model, which it can then use it to maintain a more-or-less constant flight altitude as it follows the topography, optimizing consistent overlap and GSD over the terrain.

A.5.5 Integration with Lidar

Where photogrammetry data is integrated with lidar (for point cloud colorization and/or photogrammetry), sidelap requirements will be less strict. Unless an orthophoto is required, the photogrammetric parameters may be secondary to the lidar planning. If an orthophoto is required, then the forward lap and sidelap will be needed in order to meet orthophoto requirements, and the lidar data will have more overlap and greater point density.

A.5.6 Camera Look Angle

The three basic types of photogrammetry are: true vertical photography, defined as ±0° from nadir; tilted or near-vertical photography (the most prevalent type), defined as being greater than 0° but less than ±3° from nadir; and oblique photography, when the camera is intentionally pointed off-nadir. Preferred oblique angles are defined as being between 35° degree and 55° from nadir. See Addendum VI: Best Practices and Guidelines on Mapping with Oblique Imagery.

A.5.7 Flying Height

Ensure all federal, state, and local regulations are met. In the US, Part 107 of the Federal Aviation Administration's (FAA) Code of Federal Regulations (CFR) addresses regulations pertaining to UAS flight, including allowable flying altitude without the acquisition of an exemption or waiver.

Technically, flying altitude is measured as the vertical distance above the average elevation of the area of interest. However, some UAS systems may have the ability to maintain a constant flying height over varying topography via terrain-following software.

Image GSD is partly determined by the flying height, camera focal length, and sensor pixel size. Obtaining data with a GSD higher than the minimum project accuracy should be the goal during mission planning. Planning software may adjust flight parameters, including flying height, based on desired GSD.

An increase in flying height will allow each image to cover a larger area, reducing the total number of images captured and reducing the required amount of data storage. Additionally, a sensor may require a minimum flying height to optimize output or coverage.

A.5.8 Speed of Flight

Aircraft speed may limit camera exposure and data storage. The fastest time frame that the camera can capture consecutive images is called the camera's minimum capture interval, commonly represented in images per second. Flight speed should be optimized to ensure that all images can be written to the storage device without exceeding the camera's minimum capture interval. Additionally, the camera's shutter speed will produce motion blur if the flight speed is too high, so data producers must set the flight speed accordingly. Some planning software may take these factors into account for optimal image capture.

A.5.9 Project Area Coverage

It is common practice to cover areas slightly beyond project boundaries (i.e., capturing at least two images along the flight line before and after the mapping limit), to generate a buffer. Ground control should be set toward, if not slightly beyond, the project boundary to prevent extrapolation, while still remaining within the flight coverage area.

A.5.10 Flight Line Orientation

The flight line orientation and site coverage method depends on the type of UAS vehicle platform, the topography of the terrain, and wind. For example, if the terrain has a gradual overall change in elevation from one end of the project site to the other, a fixed-wing aircraft without terrain following features may not be able to effectively adjust flying height along the length of the site and may need to fly flight lines oriented perpendicular to the direction of the elevation gradient. Wind may also affect flight line orientation and direction. There are different patterns flight lines can follow, including interlaced and parallel flight lines; whichever flight line pattern is followed, it is recommended that subsequent flight lines are flown in opposing directions, to improve camera calibration.

A.6 Flight Campaign

A.6.1 Acquisition Flight Log

It is good practice to collect and record information related to data acquisition, to use in later processing and to keep as a general record. Flight logs should collect relevant information in a consistent manner, and should always be completed per day, per project. Data producers often have a form containing fields for pertinent information, including date, location, project name, start and end times of flight, ground station instrument point ID, battery set, wind speed and direction, temperature, and any comments or other information. Flight reporting is imperative to the success of any project, as it provides essential information when investigating potential issues with the data that may crop up during processing.

A.6.2 Ground Reference Station (Base Station)

A.6.2.1 Position Dilution of Precision

Position dilution of precision (PDOP) is a numerical value which represents the quality of the project's mapping geometry. Appropriate field conditions will tolerate a PDOP of 2.0 or less, while less-than-ideal conditions should not exceed a PDOP of 6.0. Users should refer to their GNSS instrument planning software or NGS for more information.

A.6.2.2 Multipath/Signal Shading

The elevation mask of the ground reference station (or base station) should be set to an appropriate value, to ensure that adequate satellite coverage is available above the horizon. This is usually indicated by the PDOP of the resulting coverage. Refer to general static GNSS surveying practices in Addendum II.

A.6.2.3 Ground Reference Station Point

Ground reference stations are usually set up over a known point within or in close proximity to the project site. If positioned over a known point or monument, the GNSS receiver antenna should be set up and allowed to collect data for a reasonable time (usually 20 minutes or more) before and after flight, to ensure continuous collection for the duration of the flight.

If a known point is not within reasonable distance to the project area, a point may be established via a static GNSS survey. However, if an unknown point is to be established solely for the flight campaign, the static collection should be allowed to run for an hour or longer (preferably longer) both before and after flight. The point may then be established in a network with nearby Continuously Operating Reference Stations (CORS) during post-processing.

If using the National Geodetic Survey's (NGS) Online Positioning User Service (OPUS), base station collection should be observed for a minimum of 20 minutes for rapid-static solutions, or for a minimum of two hours for a static solution. Accuracies are generally improved when using OPUS static solutions.

A.7 Ground Control

Ground control accuracy should be determined according to project requirements. Refer to the ASPRS Standards for horizontal and vertical control information, including methods of accuracy assessment and checkpoints.

A variable number of surveyed ground control points should be used to reference the photogrammetric block to a projected coordinate system, while a minimum number of surveyed checkpoints should be used to validate the accuracy of the block. These ground control points are the points to which the photogrammetric block is controlled and adjusted through aerial triangulation (AT), where the block is rotated, scaled, and translated to the coordinate reference system. Although the exposure (or camera) stations act as additional control, ground control is necessary to guarantee accuracy following proper procedure as defined by the ASPRS Standards. Checkpoints are an independent collection of surveyed ground control points that validate the reported accuracy for a photogrammetric (or lidar) product and have been excluded from the processing in the aerial triangulation.

Ground control points may be surveyed or extracted secondhand from lidar, orthoimagery, or another product, provided that the error of these extracted points is taken into account relative to the desired product accuracy, as per the ASPRS Standards.

A.7.1 Number and Distribution of Ground Control Points

There is no empirical research that indicates an absolute rule regarding the number and distribution of control points for UAS photogrammetric mapping missions. It is generally accepted that the control points should be well-distributed throughout the project site for consistent geometry, and that their quantity should be proportional to the site area. The ASPRS Standards recommend a minimum required sample size of 30 well-distributed 3D checkpoints, based on the Central Mean Theorem, to report confident statistical accuracy. Refer to ASPRS Standards for more information.

Furthermore, the number of control points may be determined by a particular sensor or system. Data producers may refer to the manufacturer recommendations regarding control points, though care should be taken to seek out whether these recommendations can be supported by research, such as white papers or other literature. It is recommended to thoroughly test sensors and systems and to verify mapping product accuracies as a means of developing practical methods to achieve required project accuracies, as per the ASPRS Standards.

A.7.2 Ground Control Target Type and Size

Many styles of targets exist, and the type of ground control target may vary depending on the sensor and project criteria. A black-and-white checkerboard is a common design, but circular targets may enable more precise center location via centroid calculations. Additionally, photograph-identifiable control points (PIDs) may be used: these include paint corners, corners of varying types of impervious surfaces, and other easily identifiable features already present within the project area. The target size and shape should suit the project’s GSD, so that the target is visible in the imagery. Refer to these Standards and other literature for more information regarding ground control target shape and size.

A.7.3 Control Point Placement

Proper ground control point (GCP) placement should take vertical obstructions into account, both during the preliminary ground survey and visibility in imagery. The data producer should consider an elevation mask when determining placement. Unless it is unavoidable, control points should generally not be placed in vegetated areas. If a control point is placed atop dense vegetation such as grass, the slight elevation increase may bias the elevation in the model, causing problems in the block adjustment. If, in this instance, the grass was cleared all around the target and the point was set on bare earth, then the point would be a better representation of the ground, but it should be noted that the ground elevation outside of the cleared vegetation would still carry uncertainty. In cases like this, lidar would be a better option for the generation of surface and elevation models.

A.8 Camera Calibration

The data producer may allow the software to self-calibrate by solving the camera calibration parameters during the adjustment process. Additionally, as part of initial factory calibration, camera manufacturers may choose to provide a set of pre-calibration parameters. These factory calibration parameters may serve as useful initial values during self-calibration, but they are not always needed, as self-calibration is a common, reliable camera calibration method.

Most camera calibration models consider radial and tangential lens distortions, principal distance (i.e., true focal length), and principal point (i.e., true photo origin). Data producers should ensure that the lever arm (i.e., the distance from the point of camera exposure to the GNSS antenna reference point) is either provided by the manufacturer, solved for in the adjustment, or physically measured.

A.9 Processing

A.9.1 Processing Overview

The overall photogrammetric workflow should begin by validating the initial model via aerial triangulation (AT). The AT process includes adjusting the photogrammetric block to the projected coordinate system, then verifying that all systematic errors have been removed or minimized. Checkpoint residuals indicate whether the results are satisfactory according to project specifications. Finally, all required photogrammetric products should be generated and validated, whether it be a triangulated irregular network (TIN), a topographic map, a digital elevation model (DEM), a digital surface model (DSM), a dense point cloud, or an orthophoto/orthomosaic.

A.9.2 Image Matching

In UAS imagery processing, the first step is to perform image matching (or image stitching). The image matching software will identify and collect thousands of well-defined pixels called keypoints, and then it will identify the strongest matches between keypoints and use these to generate a sparse cloud of tie-points, by which the images become matched.

A.9.3 A Priori Standard Errors

When importing photo datasets, the software may allow the data producer to assign each photo center an *a priori* standard error in the horizontal and vertical components. Some systems may populate the exposure station standard errors with an internal estimate, which may or may not be accurate. It is recommended to initially set these values according to their published accuracies. Note that standard errors will likely be changed post-adjustment to be more or less constrained (or even excluded entirely for obvious blunders) according to their residuals.

The image matching step should approximate the locations of ground control points in their respective images for measurement. Automatic control point measurement, a feature provided in most software, may estimate control locations more precisely than the initial image matching, but it is recommended to carefully refine control measurement manually. After control is measured and assigned as either control or checkpoint, the model is then readjusted.

A.9.4 Adjusting Data

Ground control points with the desired geometrical distribution and number should be designated as control to reference the block to the project coordinate system in the bundle block adjustment. The remaining points should be designated as checkpoints. The block should then be readjusted with the desired control distribution.

Ground control residuals should be assessed and compared to their set standard errors to determine whether they are consistent with the level of accuracy at which they were surveyed. If they are not, exposure stations should be assessed to determine that they are not over-constrained and that residuals do not exceed in magnitude beyond three times their set standard errors.

If control point residuals fall within acceptable ranges and the residuals of the exposure stations are significantly below their assigned standard errors, the exposure station standard errors may then be further constrained; at this point, they are now functional as a redundant set of control points for the block and may be used for further optimization. The aim should be to have the block adjusted according to the surveyed control points, and to have the resulting statistics be based on the residuals of a reasonable sample of checkpoints. The quantity and/or distribution of control points and checkpoints may be changed if blunders occurred in the ground control survey. Excessive errors may indicate errors in control point measurement, exclusion of lever arm, over-constrained camera calibration parameters, or erroneous camera position(s), to name a few obvious possibilities.

Once AT results are satisfactory, photogrammetric products such as digital elevation models (DEMs), digital surface models (DSMs), triangular irregular networks (TINs), dense point clouds, or orthophotos may be produced. Users and data producers may refer to the ASPRS DEM User Manual for validation of these products.

A.9.5 Assessing Photogrammetric Products

Refer elsewhere in these Standards for a comprehensive guide to assessing digital mapping product accuracy.

SECTION B: BEST PRACTICES AND GUIDELINES IN MAPPING FROM UAS-BASED LIDAR

B.1 Objective

Section B of this Addendum is intended to be a concise overview of *select* technical considerations for aerial mapping using unmanned aerial systems (UAS) equipped with lidar sensors. This material has been compiled from various resources, including the Manual of Airborne Topographic Lidar (2012), the ASPRS Positional Accuracy Standards for Digital Geospatial Data (2014), the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2 (2024), the USGS Lidar Base Specifications (2022 rev. A), and the current ASPRS LAS Specifications v1.4 - R15 (2019). This publication is designed to be sensor agnostic and data-driven, without regard to a specific manufacturer or sensor type. The goals for this Addendum are to provide appropriate common guidelines and recommendations for acquiring accurate digital lidar elevation data via a topographic airborne UAS equipped with a lidar scanning system, to assist companies and agencies in establishing standards for their organizations, and to help improve efficiency for project planning and data acquisition in a clear and straightforward manner.

B.2 Introduction

UAS lidar provides extremely dense point clouds compared to traditional aerial lidar, capturing features that otherwise would not be captured by lidar scanners operating from high altitude. UAS-compatible lidar scanners represent a wide range of achievable accuracy and precision, from the lowest-grade systems to the highest. UAS lidar systems may vary in technical characteristics—such as beam quality, beam divergence, number of simultaneous channels recorded (16, 32, 64+), transmitter power, receiver sensitivity (signal-to-noise-ratio, or SNR), and wavelength—which differentiate them from systems designed for higher-altitude operation. Beam divergence, and beam quality in particular, can be of significantly lower quality for UAS lidar systems in comparison to more traditional systems.

The objective of this Addendum is to provide practical guidelines for using UAS lidar. All principles of aerial lidar apply to UAS lidar; this document covers details that pertain to collecting lidar data using a small, dynamic UAS. This Addendum includes fundamental aerial lidar mission planning and acquisition parameters that apply to UAS missions.

B.3 Platform Type

While rotary-wing aircraft are the most common platforms used on UAS lidar projects today, fixed-wing UAS platforms are becoming more widely used. Commercial vendors provide a range of fixed-wing UAS platforms, featuring both traditional and vertical-takeoff-and-landing (VTOL) options, for use as mapping platforms. As regulations open up to Beyond Visual Line of Sight (BVLOS) operations, larger commercial UAS lidar mapping projects are becoming more practical and achievable. This is particularly relevant for corridor surveys such as transmission line mapping, where rotary platforms offer lower collection efficiency.

In general, fixed-wing platforms offer greater endurance, higher altitude, and longer-range performance than comparable rotary-wing platforms. Some fixed-wing and hybrid fixed-wing VTOL UAS platforms now offer lidar payload capabilities as standard options. Rotary platforms continue to be the ideal choice for small site coverage, or in locations where takeoff and landing constraints make a fixed-wing approach less feasible.

Aside from any platform-specific setup and in-air operational requirements, guidelines for UAS lidar sensors do not vary significantly between fixed-wing and rotary-wing platforms. Mounting requirements may vary based on platform, but achievable lidar performance is generally independent of platform type. Because fixed-wing platforms usually operate at higher speeds and altitudes than rotary-wing platforms, the data producer should ensure that the lidar sensor supports an effective range and sampling density (pulse repetition frequency, or PRF) appropriate for the operational altitude and speed of the chosen UAS platform. For extended duration flights or BVLOS operations, the power requirements and collection duration of the lidar sensor should be taken into account.

B.3.1 Lidar System Specifications and Settings

Below is a review of noteworthy lidar system specifications. Most product vendors will include this information on a particular lidar system's datasheet or brochure. For a comprehensive list and explanation of the principles of lidar and aerial lidar systems, please refer to the ASPRS Manual of Airborne Topographic Lidar.

B.3.1.1 Range

The range of a lidar sensor is the maximum distance from which an incident pulse can be reflected, received, and then stored as a discrete observation. The range achievable by a lidar sensor depends on the output power and the reflectance of the target. Factors affecting range also include scan angle and object reflectance. Pulse repetition rate (PRR) is related to pulse power.

The effective range of a lidar system is dependent on the wavelength and reflectivity of the target. For mapping purposes, a system's effective range should be between 10% and 20% reflectivity target at the operational wavelength, typically in the infrared. Many UAS lidar systems are built with lidars designed for automotive applications and are calibrated to much higher reflectivity targets (80% in some cases). Care should be taken to verify range specifications for 10% (recommended) to 20% (worst-case) reflective targets.

B.3.1.2 Pulse Repetition Rate

Pulse repetition rate (PRR) is the frequency at which laser pulses are emitted by the sensor. The PRR is typically expressed in kHz, and it affects the range and point density of the data collected. Some systems feature a range of PRR settings that can be altered based on project requirements. Low PRR will result in low point density, but will increase range, while high PRR will result in high point density, but will decrease range.

Care should be taken when comparing PRR across sensors. UAS lidar sensors are often multi-channel laser transceivers, and most will indicate the total PRR rather than the per-channel PRR. In addition, many 360-degree scanners will indicate the full rotational field-of-view (FOV) as the maximum PRR; UAS mapping typically only uses 80 degrees (+/- 40 degrees around nadir) or less.

B.3.1.3 Scan Speed

Scan speed (sometimes called scan frequency) is the number of rotations/oscillations of the mirror mechanism that directs the laser pulse, typically expressed as rotations, lines, or scans per second. Like the PRR, scan speed can be adjusted in some systems. Scan speed, aircraft ground speed, and PRR determine the cross-track and along-track point spacing. A potential objective is to adjust vehicle speed, scan speed, and PRR so that cross-track spacing is approximately equal to along-track point spacing. Objectives may vary depending on project needs such as corridor mapping where along track point spacing should be more closely grouped.

B.3.1.4 Scan Pattern

Scan pattern is determined by the scanning mechanism built into the lidar system. Some scanners have multiple channels that emit multiple pulses simultaneously. Many scan mechanisms use a moving or rotating mirror, such as a rotating polygon mirror, an oscillating mirror, or a nutating mirror (elliptical). A rotating polygon mirror creates lines slightly skewed from parallel lines perpendicular to the flight direction, an oscillating mirror creates sinusoidal or zig-zag patterned scan lines, and a nutating mirror creates circular arrays of points. Different scan patterns can be advantageous for certain applications. For example, a scan pattern from a nutating mirror may be well-suited for corridor mapping such as overhead power line mapping, but it would be inefficient for wide area topographic mapping, where a rotating polygon or sinusoidal mirror would be preferable.

B.3.1.5 Field of View

The field of view (FOV) is a measure of angular sweep of the laser channel. FOV range is measured either with respect to nadir, or as a total angular sweep. Systems with multiple channels or with special designs may also have an along-track FOV, due to the spread of the multiple beams. This is typically on the order of 1–2 degrees per channel, so they can scan both forwards and backwards with respect to the direction of flight as well as at nadir.

B.3.1.6 Instantaneous Field of View

When planning, it is important to know the size of a beam footprint when projected on the ground. Knowing beam footprint size helps determine the optimal point spacing and allows the data producer to ensure that the entire area is covered and that there are no gaps in the data. Beam footprint is determined by the instantaneous field of view (IFOV) and is given as either milliradians or as millimeters per 100-meter range. High altitude lidar systems typically have a symmetrical beam divergence with a near-circular beam footprint, while UAS lidar systems often have an asymmetric beam divergence with a major and minor axis to the resulting elliptical or rectangular beam footprint. UAS lidars, especially entry-level systems, can have beam divergences that are an order of magnitude larger than traditional lidar systems, which can be advantageous as large beam diameters can capture returns from features such as overhead power line conductors more easily than beams with smaller diameters; however, larger beam footprints introduce more uncertainty/error in to the XY location of the return, and they reduce the resolution on linear features which are smaller than the beam size, introducing "fuzziness".

B.3.1.7 Multiple Pulse Returns/Echoes

Multiple observations or points along the path of a single pulse are called returns or echoes. Modern lidar sensors are capable of capturing multiple returns per pulse, which is advantageous when ranging through relatively dense vegetation. Additionally, when a pulse penetrates through vegetation, the final return does not always represent the ground; therefore, the use of "last pulse" logic in the receiver—essentially ignoring intermediate returns and waiting for the last echo to be received—further improves canopy penetration capability and should be enabled when mapping under vegetation. Data can also be filtered to display points by return number, which may help in lidar point classification. Note that if the ground cannot be seen through the vegetation by the naked eye, a pulse may not be able to penetrate through to the ground at all.

B.4 Mission Planning

B.4.1 Safety and Regulations

For regulations on allowable flying altitude for small, unmanned aircraft systems, consult Part 107 of the Federal Aviation Administration's (FAA) Code of Federal Regulations (CFR). These regulations must be followed at all times unless an exemption or waiver has been granted. Ensure all federal, state, and local regulations are met when operating a UAS.

Safety is essential to any UAS mission and includes awareness and reconnaissance of tall objects or obstructions. Potential hazardous obstructions may ultimately dictate the minimum flying height that is still compliant with FAA rules, or that piloted aircraft be used instead of a UAS.

B.4.2 Weather and Other External Factors

Leaf-off conditions are ideal for lidar data acquisition. However, wet ground or ground covered in snow or ice will introduce issues in surface and object reflectance. This can cause ranging errors or absorbed pulses, ultimately resulting in gaps in the data. Fog, rain, smoke, or dense clouds can also cause ranging errors, as well as excessive noise and atmospheric clutter in the data.

B.4.3 Acquisition Parameter System Settings

B.4.3.1 Point Spacing

Achieving accurate, well-distributed points is the objective when surveying with lidar. The PRR, flying height, flight speed, scan speed, scanning pattern, and swath overlap combine to develop the desired point distribution and density.

In scan patterns created by rotating and oscillating mirrors, point spacing is measured in two directions, cross-track, and along-track. For topographic mapping, the goal is to make the spacing in both directions approximately equal. Cross-track point spacing is determined by the PRR, flying height, and scan speed, while the along-track point spacing is determined by the scan speed, flying height, and flight speed. Along-track point spacing is optimized when the zig-zag pattern of oscillating mirror scanners form interference patterns (i.e., alternating and opposing zig-zag patterns).

B.4.3.2 Field of View

The swath width is determined by the field of view (FOV) combined with the flying height. In ideal scenarios like flat terrain, FOV values are typically around 45 degrees from nadir. In flat terrain, ranging errors are more likely to occur at wide angles due to extreme angles of incidence. In areas with more vertical features (including dense vegetation), FOV values are typically around 40 degrees from nadir.

B.4.3.3 Overlap

Unlike photogrammetry where overlap is necessary to compute a 3D point, lidar is an active sensor and therefore does not require overlap. However, it is good practice to plan for overlap, since having overlapping swaths will increase point density, allow for relative accuracy assessments, and can be used in system calibration. More than 50% overlap is ideal, as this provides additional flexibility: points at swath edges are often associated with large scan angles and can be eliminated or filtered while maintaining point density. Additionally, swath-to-swath lidar registration computations benefit from overlap data via tie-planes.

For wide-area mapping where terrain-following is used for the flight, swath overlap should fall between 25% and 35%. Plan for overlap in corridor mapping to increase point density, even if a single pass will cover the area. Note that overlap can be negatively affected by undulating terrain; varying topographical terrain with large reliefs can prevent constant overlap when flying at fixed altitude above MSL. In extreme cases, plan for higher overlap to avoid data gaps.

B.4.3.4 Flying Height and Flight Line Spacing

Overlap is determined by the flight line spacing combined with the flying height and FOV. In flat terrain, a good starting point is to make the flight line spacing equal to the altitude, with an FOV of 45 degrees.

B.4.3.5 Flight Line Speed and Orientation

Ideal flight speed is a balance between the desired point spacing and aircraft power usage. Wind speed and direction can affect the aircraft’s ground speed and aircraft orientation. Headwinds with greater speed than the aircraft reduce ground speed, which impacts along-track point spacing.

The flight line orientation and coverage pattern depend on the topography of the terrain, as well as wind conditions. If the area is comprised of a large slope, it is good practice to fly perpendicular to the slope direction, as this will avoid excessive power consumption by maintaining a constant altitude across each flight line.

When accounting for wind conditions, flying perpendicular to the wind direction not only avoids excessive power usage in headwinds, but also mitigates the wind’s effect on aircraft pitch and speed. Changes to pitch and speed can negatively affect point spacing, making passes overly dense in headwinds and excessively sparse in tailwinds.

B.4.4 Onboard Positioning and Direct Georeferencing Systems

Direct georeferencing systems are made up of a number of essential components, including inertial navigation systems, lidar sensors, and software. Direct georeferencing with lidar follows three basic steps: (1) the GNSS records 3D positions while the IMU records the roll, pitch, and yaw of the sensor, (2) the raw airborne vehicle trajectory is refined to create Smooth Best Estimate of Trajectory (SBET), and (3) the SBET is combined with the range, azimuth, and elevation data to compute the point positions.

When direct georeferencing for precision UAS lidar, the GNSS receiver should be capable of dual-frequency L1/L2 channels (also known as survey-grade GNSS signal receiving) at the minimum. Aircraft may be equipped with two GNSS receivers, one for aircraft navigation to follow waypoints laid out during flight planning, and the other for precise positioning. Precise point positioning (PPP) or post-process kinematic (PPK) are typically used for aircraft trajectory correction; Real-Time Kinematic (RTK) may be used, but is less common for precise UAS lidar, as it is generally more suitable for rapid-response applications.

B.5 Control Network Design

Ground control points represent the surveyed network to which the point cloud is translated vertically and are used to adjust the point cloud to a projected coordinate system. Checkpoints, by contrast, are an independent collection of surveyed ground control points which are not used to reference the point cloud to the network, and which serve as the source for the reported accuracy.

While there is no empirical research to indicate the best possible number and distribution of control points for lidar flight campaigns, control points should be well distributed throughout the project site in respect to terrain geometry, and their quantity should scale proportionally to the size of the site.

For more information regarding control and checkpoint accuracy to meet product accuracy, please refer to the ASPRS Standards.

B.6 Considerations During Day of Flight Campaign

B.6.1 Flight Log

Flight logs are standard practice for tracking relevant information throughout the project. Flight logs may be printed or digital, and typically include the date, location, project name, start and end times of each flight, ground station instrument point ID and serial number, battery set, wind speed and direction, and temperature, as well as any other relevant information or comments.

B.6.2 Ground Reference Station

B.6.2.1 Multipath/Signal Shading

The ground reference stations (or base stations) are typically placed in a location with minimal vertical obstructions or other objects that could cause multipath and/or GNSS signal shading.

For more information about field surveying techniques, please refer to Addendum II: Best Practices and Guidelines for Field Surveying of Ground Control Points and Checkpoints to the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, v. 2 (2024).

B.6.2.2 Position Dilution of Precision

Position Dilution of Precision (PDOP) is a numerical value representing the integrity of the geometric quality of GNSS satellites configuration at a specific location and time. Generally, a PDOP of 3.0 or less is ideal. It is recommended to use GNSS planning software to view projected PDOP for acquisition of an area of interest (AOI) at a specified date and time.

B.6.2.3 Collection Frequency Minimum (Consistent with Receiver Paired with IMU)

The static base station and the GNSS receiver of the sensor (not the aircraft) must collect data at a minimum frequency of 1 second. If the collection frequency is longer, the platform can travel a considerable distance before collecting new data; for example, if the GNSS receiver has a collection frequency of 5 seconds, and the platform is travelling at a speed of 5 m/s, the platform will have travelled 25 meters between recorded positions. Therefore, trajectory positions anywhere between collections would be imprecisely interpolated.

B.6.2.4 Selection of the Station Points

As per the Manual of Airborne Topographic Lidar, the ground reference station should be set up on an "established or surveyed monument such as those published by NGS or USGS." If the station is set up over a known point or monument, the GNSS receiver antenna should be collecting for a minimum of fifteen minutes before and after acquisition, to ensure the GNSS data covers the entire collection duration—an essential component for successful trajectory calculations during post-processing.

If a known GNSS control point is not within a reasonable distance to the project area, one may be established via a static GNSS survey. However, if an unknown point is to be established solely for the flight campaign, the static collection should be set up and permitted to collect for at least two hours both before and after flight. The point may then be established in a network with nearby Continuously Operating Reference Stations (CORS) or similar network during post-processing. The point may also be established via the National Geodetic Survey's (NGS) Online Positioning User Service (OPUS).

B.6.2.5 Field Calibration and IMU Alignment

Boresight calibration is typically resolved by the manufacturer, but it is good practice to check calibration prior to each mission. IMU alignment should be performed before and after flight. There are two types of IMU alignment: static alignment (also known as course calibration), and dynamic alignment (also known as fine calibration). Static alignment occurs during system startup and sets a rough cardinal heading. For this reason, a manufacturer may recommend allowing the system to rest in a stationary position for anywhere from 30 seconds to 5 minutes before and after flight.

Dynamic alignment varies from system to system, and the data producer should refer to manufacturer recommendations. This type of alignment is performed after takeoff during aerial maneuvers and ensures that the IMU heading does not experience drift by making fine adjustments to the system's heading. While exact procedure may vary, some systems make dynamic adjustments by flying in a figure-8 pattern, followed immediately by flying in an S pattern as the aircraft makes its way back to the starting flight line. For long flight lines, dynamic alignment may also need to be performed at certain intervals as per manufacturer recommendation to prevent drift along the flight line.

Both static and dynamic alignment procedures should be performed before and after each flight, bookending the data collected. At least two cross-flight scan patterns should be flown to aid in postflight system calibration. For more information about system calibration, please refer to the system manufacturer.

B.7 Data Quality Control

B.7.1 Error and Bias

Sources of error and bias are to be understood, resolved, and explained as best as possible prior to accuracy assessment. Error in lidar data can result from GPS errors or IMU drift over the course of a flight or multiple flights. These errors can result in line-to-line or flight-to-flight mismatches in the data. A qualitative assessment of line-to-line and flight-to-flight matching can determine where and to what extent errors exist within a dataset. A strip-alignment software package should be used to resolve mismatches.

B.7.2 Vertical Debiasing

Once the vertical accuracy of the lidar data has been assessed for accuracy, the point cloud may be vertically translated according to the value of the mean vertical error (note that this process is only valid if the standard deviation of the mean is small compared to the value of the mean). This calibration process “bumps” the dataset up or down to remove vertical bias that was not completely resolved by the trajectory post-processing software during direct georeferencing.

B.7.3 Assessing Horizontal Shifts

Because each point in the point cloud is discrete, assessing horizontal accuracy of lidar can be challenging and uncertain. However, an obvious horizontal shift may indicate a mistake in processing or some other type of systematic error. The use of large aerial targets for GCPs, similar to those used in photogrammetry, can help reveal potential incidents of horizontal bias in the dataset. Target style and size should be determined by point density.

Once the data has been collected, the data producer can sort the data by intensity value, and then navigate to each aerial target within the project to compare each target’s actual position to its surveyed location. Then, the data producer can check and see if there are commonalities in the direction and distance of a shift in all target locations. A uniform shift could indicate that the surveyed control is in the incorrect coordinate reference system (i.e., local vs. state plane, or grid as opposed to ground), or that an error was made in processing (e.g., the project itself is set up in the incorrect coordinate reference system or has incorrect units).

One approach to check horizontal accuracy is to make a comparison between elevation profiles derived from the data and a reference surface. Another approach is to overlay a photogrammetric product of the area over the collected data. For more information regarding horizontal accuracy assessment of lidar, please refer to the ASPRS Standards. Another useful reference is the Absolute 3D Accuracy Assessment of UAS LIDAR Surveying by Lassiter, et al. (2021).

B.7.4 Control Point and Targeting Styles

In lidar data, control points for data calibration and adjustment do not need to be visible in the point cloud because only elevation is of importance. However, it is important to assess the horizontal position of the point cloud, as this can indicate other issues. Factors such as flying height and point density determine the size and type of aerial targets. Some examples of targets are listed:

1. Black and white checkerboards (e.g., 60 cm x 60 cm)
2. Flight crosses
3. Chevrons (2ft length, 1ft wide)
4. 5-gal orange or white bucket lid
5. Existing terrain features

Figure V.B.1 shows examples of different target designs for GCPs used by the UAS community for photogrammetry and/or lidar projects:

Figure V.B.1 Ground Control Point Designs (courtesy: Dr. Abdullah and Dr. Munjy workshop "Aerial Triangulation and Data Processing for the UAS")

B.8 Data Processing

Included in this section are practices for responsible data handling and contract fulfillment, as well as a general overview for data processing and quality assurance/quality control (QA/QC). A thorough processing workflow incorporates QA/QC at multiple stages, to detect and resolve problems efficiently.

B.8.1 Project Preparation and Pre-processing

Immediately archive all raw lidar, GNSS, and IMU data, creating duplicate copies for subsequent processing. Organize all essential raw data and reference files, such as buffered project boundaries, surveyed ground control, and flight logs, following an appropriate file structure. To facilitate project preparation, adhere to the recommended pre-processing steps outlined below:

1. Review project specifications and accuracy requirements.
2. Confirm the horizontal and vertical coordinate reference system.
3. Convert GPS observations from local base stations to the Receiver Independent Exchange Format (RINEX).
4. Process the raw trajectory to derive the Smooth Best Estimate of Trajectory (SBET) and thoroughly examine relevant QA/QC reports.

5. Trim flight lines to eliminate unnecessary segments, including turns, transit lines, or other in-air maneuvers not essential for site coverage. Reference buffered project boundaries to assist in this trimming process.
6. Merge the SBET with raw lidar data to generate an uncalibrated, unclassified point cloud.

B.8.2 Validation and Calibration

After completing pre-processing, the data must undergo a final calibration process for control and validation, adhering to specifications outlined in the contract documents and reference files (e.g., surveyed ground control, buffered project boundaries, etc.). Failure to meet contract requirements and project specifications may result in rejection and the need for re-flights. The following steps are recommended:

1. Review project specifications and accuracy requirements.
2. Verify Area of Interest (AOI) coverage against buffered project boundaries, ensuring completeness and adequate point density.
3. Conduct qualitative line-to-line and flight-to-flight analysis to identify and resolve strip matching issues.
4. Verify boresight alignment.
5. If necessary, utilize strip alignment software to eliminate any systematic misalignment, such as residual boresight error, between flight lines.
6. Calibrate lidar-to-ground control.
7. For excessively large datasets, export calibrated lidar using a tiling scheme.

B.8.3 Post-processing

The following steps summarize best practices for the classification and QA/QC of a point cloud:

1. Review contract documents for classification scheme and final deliverables.
2. If classifying macros are used, test on sample tiles before processing in batch.
3. Perform visual inspection for QA/QC.
4. Perform manual review and cleanup of classified features, as necessary.

The above steps will vary based upon the data producers’ specific workflows and project requirements.

ADDENDUM VI: BEST PRACTICES AND GUIDELINES ON MAPPING WITH OBLIQUE IMAGERY

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FIGURES

TABLES

OVERVIEW

This document provides definitions, explanations, and recommended specifications necessary to properly quantify various properties of digital aerial oblique imagery.

This document seeks to inform and guide the user in the following ways:

- Provide clear and updated definitions of concepts and terms relating specifically to digital aerial oblique imagery.
- Provide a framework of quantifiable best practices and guidelines for defining the technical details of digital aerial oblique imagery products.

PURPOSE

The American Society for Photogrammetry and Remote Sensing (ASPRS) is the leading scientific professional organization representing the photogrammetry and remote sensing profession. With the dramatic increase of digital aerial oblique imagery capture platforms, products, and services, ASPRS recognizes the need to establish consistent language to describe the various capture platforms, products, and services so that purchasers and end users can compare these widely varying items appropriately.

This Addendum represents the best efforts of ASPRS to define and clarify the key issues that affect the analysis and comparison of digital aerial oblique imagery products and services.

This Addendum was prepared by the Oblique Imagery Guidelines Working Group, an ad hoc group formed by the ASPRS Primary Data Acquisition Division. The group's core members included representatives from the commercial and private sectors, sensor manufacturing, and academia. During the development of this document, the group interviewed hardware and software manufacturers, procurement representatives from state and federal agencies, and private providers of commercial geospatial mapping products.

This Addendum intends to offer providers, procurement personnel, and end users a resource to help distinguish between various digital aerial oblique imagery products and services.

This Addendum provides pertinent definitions to highlight the characteristics distinguishing digital aerial oblique imagery from traditional nadir or orthogonal imagery products and services. This document describes methods for the planning of standard flight missions for typical oblique sensors. Other sensor types and project types are not discussed at length, but may be addressed in future updates. Reporting requirements for all types of projects and sensors are defined and explained in this document.

SCOPE

This document will define best practices and guidelines for the imagery captured by digital frame cameras from manned aircraft and unmanned aerial systems (UAS) (see Addendum V: Best Practices and Guidelines on Mapping with Unmanned Aerial Systems (UAS)). The Guidelines do not apply to push broom/whiskbroom sensors or spaceborne cameras. This Addendum considers the use of all types of digital frame cameras, including but not limited to: calibrated or uncalibrated photogrammetric quality sensors; consumer or professional cameras; and multispectral, hyperspectral, and thermal cameras. However, the primary focus of this Addendum will be the typical oblique camera system and standard oblique projects, as defined below.

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TERMS AND DEFINITIONS

- *along-track oblique* – Forward and/or backward-looking oblique images; oblique images looking along or opposite to the direction of flight or the forward/aft axis of the aerial platform.
- *apparent horizon* – The visible line between the terrain and the sky. If the terrain is perfectly level, the apparent horizon will appear slightly curved near the upper edge of the photograph[1]. See Figure VI.1 also.
- *average project oblique GSD* – A value representing the overall oblique project ground sample distance (GSD). This is separate from any nadir GSD value and must specify the location within the oblique image where the value of the GSD is estimated. See Section B.1.4 for the method of calculation.
- *backward-looking* or *rear-looking oblique* – An oblique aerial image captured looking behind the direction of movement of the camera.
- *capture event* – The point in space occupied by the lens system of a camera at the moment of exposure. Also called an exposure station, camera event, or capture station.
- *center line* – The shortest distance between the near and far lines of an image, as measured along the line containing the center point.
- *depression angle* or *true depression angle* – The angle measured in the principal plane between the true horizon and the camera axis[2]. See Figure VI.1 also.

[1] *Photogrammetry*, Francis Moffitt, 1967
[2] *Photogrammetry*, Francis Moffitt, 1967

- *dip angle* – The angle measured in the principal plane between the true and apparent horizons[1]. See Figure VI.1.
- *dynamic range* – A representation of a capture system's ability to record the brightness range of a scene; alternately, the limits of luminance range that a digital camera can distinguish in a single image at extremes of bright and dark. This differs from quantization, which defines the number of gradations of unique luminance that can be distinguished.
- *exposure station* or *camera station* – The location of the camera in space at the moment of image capture.
- *far line* or *backline*– The rearward-most edge (farthest from the camera station) of an oblique image. In a high oblique image, the far line approaches infinity. See Figure VI.B.2.
- *far-oblique region* – The region of an oblique perspective image from the ¾ line to the far line. See Figure VI.B.2.
- *foreshortening* – In aerial oblique imagery foreshortening refers to the distortion of objects or features due to the camera's angle relative to the ground. When capturing images from an oblique angle, objects closer to the camera appear larger, while those farther away appear smaller. It also highlights the scaling of a pixel projected onto flat terrain (stretching along line of sight) or vertical building facades.
- *forward-looking oblique* – An oblique aerial image captured looking ahead of the direction of movement of the camera.
- *ground sample distance (GSD)* – The distance on the ground of the smallest discrete unit of measurement within an image in the X and Y components. In GSD reporting for oblique aerial imagery, the location within the frame must be defined as the third parameter. These Standards define the GSD as, "*The linear dimension of a sample pixel's footprint on the ground. In raw imagery, pixel size is not uniform and varies based on sensor orientation and terrain. The term "nominal GSD" refers to the average or approximate size of pixels in raw imagery. In orthorectified imagery, the GSD for all pixels is uniform and constant regardless of the terrain variation".*
- *high oblique image* – An oblique aerial image that contains the horizon.
- *image nadir* – The point at which a vertical line through the perspective center of the camera's lens system intersects the plane of the image. In oblique aerial imagery sensors and sensor systems, the term "nadir" may refer to the image of a typical camera system with no tilt displacement (i.e., a standard vertical aerial mapping image). Also called the nadir and the nadir point.
- *image scale variation* – In an oblique aerial image captured by a rectangular or square CCD or CMOS sensor array, the ground coverage associated with each pixel is roughly trapezoidal. With an oblique image frame presented as a perspective image, the scale is not corrected and is

[1] *Elements of Photogrammetry*, Paul Wolf, 1974

proportional to the angular orientation of the tilted camera frame. Therefore, in an oblique aerial image, the apparent size of an object on the ground will vary as a function of its position in the image frame.

- *individual oblique frame GSD* – The computed GSD value of an individual oblique imagery frame at a specific point in the frame.
- *left line* – With an oblique aerial image oriented such that when the far line is at the top of the frame, the left line of the oblique aerial image is to the left of the near line. See Figure VI.B.2.
- *left oblique* – An oblique aerial image captured looking to the left of the direction of movement of the camera.
- *look angle or off-nadir angle or tilt angle or angle of obliquity* – The angle measured in the principal plane between the camera axis and the vertical line through the exposure station[1]. See Figure VI.1. These terms all refer to the angle at which an aerial imaging system captures an image relative to the nadir. These terms are often used interchangeably, although they may have specific nuances depending on the context or application.
- *low oblique* – An oblique aerial image that does not contain the horizon.
- *midline* – In an oblique image, the line parallel to and equidistant from the image's near line and far line.
- *mid-oblique region* – The region of an oblique perspective image from the ¼ line to the ¾ line. See Figure VI.B.2.
- *near line* or *frontline*– The leading edge (nearest to the camera station) of an oblique image. See Figure VI.B.2.
- *near oblique region* – The region of an oblique perspective image from the near line to the ¼ line. See Figure VI.B.2.
- *oblique aerial image* – An aerial image taken where the camera axis is intentionally tilted from the camera's vertical axis.
- *overlap angle* – The angle from the individual oblique aerial image to the next successively captured oblique aerial image. See Section F.3 for a complete description of the overlap angle calculation for specific look angles. Also called overlap or overlap percentage.
- *perspective image* or *discrete image* or *image frame* – A georeferenced digital image in which displacement of objects in the image due to tilt, terrain relief, sensor distortions, and orientation have not been removed. Contrast with an ortho-rectified image where sensor distortions, terrain relief, and orientation have been removed using a digital terrain model of the ground.

[1] Elements of Photogrammetry, Paul Wolf, 1974

- *pushbroom scanner* – A linear array that captures successive lines of image data of one-pixel depth as the sensor moves forward along the direction of flight.
- *right line* – With an oblique aerial image oriented such that when the far line is at the top of the frame, the right line of the oblique aerial image is to the right of the near line. See Figure VI.B.2.
- *right oblique* – An oblique aerial image captured looking to the right of the direction of movement of the camera.
- *side oblique* or *lateral oblique* – Oblique images looking perpendicular to the direction of flight or the forward/aft axis of the aerial platform. Left and right obliques are side obliques.
- *standard aerial oblique project dataset* or *standard dataset* – A dataset created by taking oblique images representing all parts of a project area in the four cardinal directions (North, South, East, and West), then pairing these images with a nadir image in order to generate an orthomosaic. This dataset may be acquired using a typical oblique camera system (as defined below), but it can be acquired in any manner that produces an image set containing oblique images in the four cardinal directions, plus an associated nadir image.
- *true horizon* – In a high oblique, the photographic trace of a horizontal plane containing the exposure station; specifically, the intersection of the horizontal plane with the plane of the image[1]. See Figure VI.1 also.
- *typical oblique camera system* – See Section A.1 for a description of a typical system as used in this Addendum.
- *vertical image* or *nadir image* – An image captured from a nominally straight-down perspective.
- *whiskbroom scanner* – A sensor that collects a single pixel at a time by sweeping in a direction perpendicular to the flight path, collecting a one-pixel depth across a flight line, then advancing in the direction of flight to collect the next row of pixels.

[1] *Photogrammetry*, Francis Moffitt, 1967.

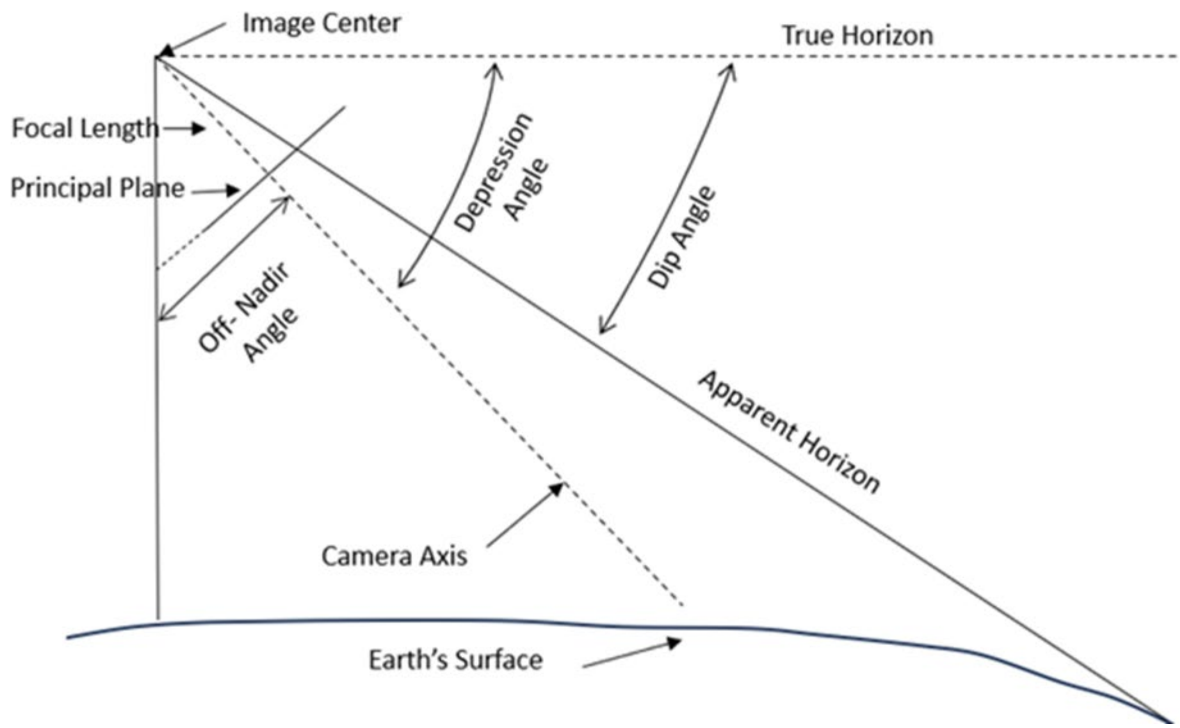

Figure VI.1 Oblique Imagery Angle Definitions; adapted from Elements of Photogrammetry, Paul Wolf, 1974.

SECTION A: CAMERA SYSTEM

A.1 Camera Description

A typical camera is a five-camera system with one nadir-oriented camera and four oblique-oriented cameras. The oblique cameras are mounted relative to the nadir camera such that there are left-, right-, forward-, and backward-looking oblique images on a flight line. All the cameras are triggered simultaneously or near-simultaneously to create a single capture event.

Systems built with other designs (more/less oblique views, single cameras, etc.) in addition to the required nadir camera are no less valuable and can produce data of the same quality; however, to avoid misunderstanding in reporting, an atypical oblique camera system should be identified and agreed upon between the data user and data producer prior to pricing the project or signing a contract. Users of such atypical camera systems should adapt flight planning, reporting, and other aspects outlined in this Addendum accordingly.

A.2 Camera Calibration and Boresight

For a typical oblique camera system, an initial calibration (lab or *in situ*, see Addendum III for more details) should be performed to manufacturer specifications and the results should be appropriately applied in the production software. Subsequent calibrations should be scheduled according to the camera manufacturer's recommendations. Analytical camera self-calibration during data processing and aerial triangulation (AT) are acceptable forms of camera calibration for subsequent calibrations when of appropriate size and quality with sufficient ground control. Self-calibration is also recommended for the processing of individual datasets.

Additionally, a proper boresight of the camera systems following IMU/GNSS manufacturer guidelines to determine the rotation matrix of the IMU to the camera (usually the nadir in a typical camera system) should be performed whenever a change in the mounted camera positions or lever arms is expected. This is especially necessary when no ground control or post-processing techniques are used and the IMU/GNSS is the only source of imagery positioning. IMU/GNSS post-processing should incorporate these boresight angles in the solution.

SECTION B: OBLIQUE IMAGING SYSTEM GEOMETRY

Figure VI.B.1 illustrates the geometry of an oblique imaging system comprised of one nadir camera and one oblique camera at angle β from nadir. The figure illustrates the oblique image ground coverage and how such coverage changes from the image near side point A• to the image far side B•, resulting in variable GSD as the coverage moves from A• to B•.

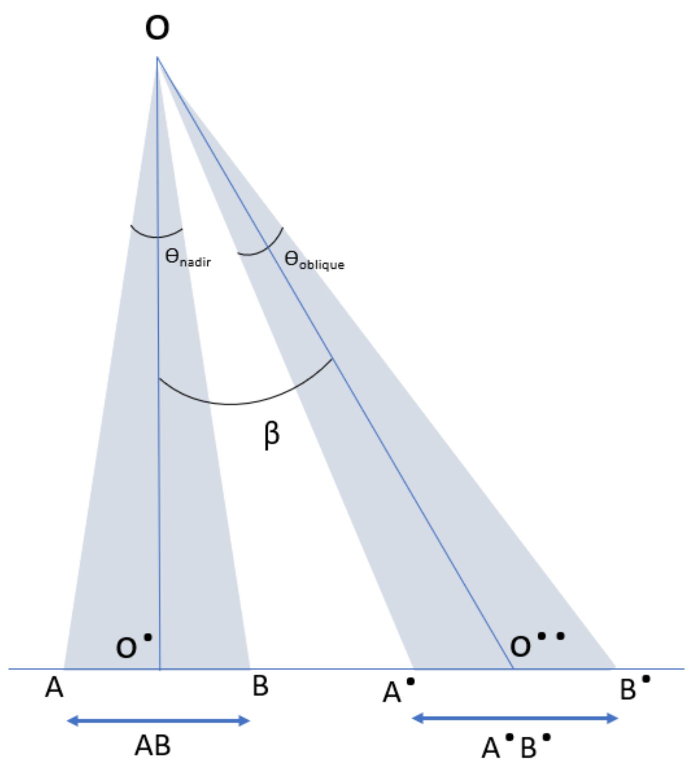

Figure VI.B.1 Geometry of an Oblique Imaging System (courtesy: Dr. Qassim Abdullah)

B.1 Ground Sample Distance

Due to the variable scale of an oblique image, reporting ground sample distance (GSD) is one of the challenges of oblique imagery datasets. Below are guidelines and methods for determining and reporting an oblique image's GSD (both planned and actual).

B.1.1 GSD Background

Although the term GSD is used for both nadir and oblique imagery, the meaning will vary between those two contexts. Due to factors such as sensor tilt and terrain relief, the scale of an image may vary as a function of pixel location within the image frame. This variation exists in both oblique and nadir images. In an orthogonal image, such scale variance is removed through the orthorectification process.

B.1.2 Computing GSD for Oblique Image Frames

During the project planning stage, it is important to understand how to compute the GSD for an oblique pixel. It is beneficial to know how to compute the GSD for both the oblique imagery and the vertical imagery, as these computations are related and differ only by the effect of the camera look angle (β). To illustrate these computations, two different scenarios are discussed:

B.1.2.1 Computing GSD When Oblique and Nadir Cameras Are Identical

Although most oblique systems available on the market use two different camera configurations for oblique and nadir imagery, this example is meant to help the reader understand the effect of the oblique geometry on the varying ground sample distance (GSD) across the image. Assume a scenario using a system with the following parameters:

Lens principal distance = 120 mm

CCD size = 0.005 mm (or CMOS size)

Lens total field of view (FOV) (Θ) = 25 degrees

Look angle (β) = 45.00 degrees (off-nadir)

Flying altitude (AMT) = 2500 m (AMT)

B.1.2.1.1 Computing Resolution Across Line-of-sight

Using the formulas in Section VI.B.1 of Appendix VI.B, the following can be calculated:

$$\mathrm{GSD_{nadir}} = \left(\frac{0.005}{1000}\right) \times \left(\frac{2500}{\left(\frac{120}{1000}\right)}\right) = 0.104 \text{ m}$$

$$\mathrm{GSD_{near\text{-}oblique}} = \frac{\mathrm{GSD_{nadir}}}{\cos\left(45.00 - \left(\frac{25}{2}\right)\right)} = 0.124 \text{ m}$$

$$\mathrm{GSD_{mid\text{-}oblique}} = \frac{\mathrm{GSD_{nadir}}}{\cos(45.00)} = 0.147 \text{ m}$$

$$\mathrm{GSD_{far\text{-}oblique}} = \frac{\mathrm{GSD_{nadir}}}{\cos\left(45.00 + \left(\frac{25}{2}\right)\right)} = 0.194 \text{ m}$$

Table VI.B.1 lists the resulting GSD across line of sight for camera mounting angles of 25.0, 30.0, 40.0, 45.0, and 50.0 degrees off nadir.

Table VI.B.1 Changes in across-line-of-sight GSD by camera look angle (β) and the location within the Oblique Image (courtesy: Dr. Qassim Abdullah)

Image Resolution (m)	Camera look angle β, (deg.)/ GSD (m)				
	25.0	30.0	40.0	45.0	50.0
Distance to Ground	2758.4	2886.8	3263.5	3535.5	3889.3
At nadir	0.104	0.104	0.104	0.104	0.104
Min oblique (front line) resolution (m)	0.107	0.109	0.117	0.124	0.131
Mid oblique resolution (m)	0.115	0.120	0.136	0.147	0.162
Max oblique (back line) resolution (m)	0.131	0.141	0.171	0.194	0.226
% change in GSD (Max./Min.)	126%	136%	164%	186%	217%

B.1.2.1.2 Computing Resolution Along Line-of-sight

Using the formulas in section VI.B.2 of appendix VI.B, one can calculate the GSD along the line of sight which practically represents the oblique GSD we are after.

Position A• of Figure VI.B.1: $\mathrm{GSD}_{\text{near-oblique}} = \left(\frac{\left(\frac{0.005}{1000}\right)}{\left(\frac{120}{1000}\right)} \times \frac{2500}{\cos\left(45-\frac{25}{2}\right)}\right) / \sin\left(90-\left(45-\frac{25}{2}\right)\right)$ = 0.146 m

Position O•• of Figure VI.B.1: $\mathrm{GSD}_{\text{mid-oblique}} = \left(\frac{\left(\frac{120}{1000}\right)}{\left(\frac{120}{1000}\right)} \times \frac{2500}{\cos(45)}\right) / \sin(90-45)$ = 0.208 m

Position B• of Figure VI.B.1: $\mathrm{GSD}_{\text{far-oblique}} = \left(\frac{\left(\frac{0.005}{1000}\right)}{\left(\frac{120}{1000}\right)} \times \frac{2500}{\cos\left(45+\frac{25}{2}\right)}\right) / \left(90-\left(45+\frac{25}{2}\right)\right)$ = 0.361 m

Table VI.B.2 lists the resulting GSD along line of sight for camera mounting angles of 25.0, 30.0, 40.0, 45.0, and 50.0 degrees off nadir.

Table VI.B.2 Changes in Oblique along-line-of-sight GSD by camera look angle (β) and the location within the Oblique Image (courtesy: Dr. Qassim Abdullah)

Image Resolution (m)	Camera look angle β, (deg.)				
	25.0	30.0	40.0	45.0	50.0
Distance to Ground	2758.4	2886.8	3263.5	3535.5	3889.3
GSD at nadir (m)	0.104	0.104	0.104	0.104	0.104
Min oblique (front line) resolution (m)	0.109	0.115	0.132	0.146	0.165
Mid oblique resolution (m)	0.127	0.139	0.178	0.208	0.252
Max oblique (back line) resolution (m)	0.165	0.192	0.281	0.361	0.489
% change in GSD (Max./Min.)	159%	184%	270%	346%	469%

Table VI.B.2 illustrates how the GSD of oblique imagery deteriorates as the look angle off nadir increases. In the example, the GSD at the far line of the oblique image is larger than the GSD at nadir by about 346% for the camera system and parameters used in the example. Such deterioration in the oblique GSD forced the manufacturers of oblique imaging system to select different cameras for the nadir and oblique imagery capture which is illustrated in the example in section B.1.2.2.

B.1.2.2 Computing GSD When Oblique and Nadir Cameras Are Different

To minimize the difference between the oblique and nadir GSD, manufacturers creatively use an oblique camera with a longer principal distance than for the nadir camera.

When the formulas in Appendix B are used to compute the GSD across the imagery coverage for an oblique imaging system with the following project parameters:

Nadir camera principal distance = 60 mm

Nadir camera FOV = 60 degrees

Oblique camera principal distance = 150 mm

Oblique camera FOV = 20 degrees

CCD/CMOS size = 0.0036 mm

Oblique angle (β) = 45 degrees

Required GSD_{nadir} = 7.5 cm

the following values for the GSD and the flying altitude are obtained:

From nadir camera:

$$\text{Flying altitude } = \frac{GSD}{\mathrm{CCD_{size}}} x\ focal\ length = \frac{0.075}{\mathit{0.0036}} x\ 60 = 1250\ m$$

From oblique camera:

$GSD_{near\text{-}oblique}$ = 4.5 cm

$GSD_{mid\text{-}oblique}$ = 6.0 cm

$GSD_{far\text{-}oblique}$ = 9.1 cm

With such systems, the oblique GSD at the far oblique region of the image of 9.1 cm is only deteriorated by 122% when compared to the GSD at nadir which is equal to 7.5cm. As a matter of fact, such system configuration resulted in oblique imagery with higher resolution, 6.0 cm, than the nadir imagery at the mid-oblique region. Manufacturers can design camera configuration to make the oblique GSD equal to the nadir GSD or even higher resolution at the desired look angle.

B.1.3 Representative GSD for Individual Oblique Image Frames

When reporting the GSD for a single oblique image frame, the GSD can be measured at either the center point or the principal point of the individual image frame (see Figure VI.B.2). This center point of the image is the typical GSD reporting location for oblique image frames. Note that if the GSD is estimated or calculated via GNSS or post-processing using a height model for calculation, the source of the height data should be included.

If desired, other GSDs may be reported for locations within the image at the near line, ¼ line, ¾ line, or the far line of the image frame (see Figure VI.B.2). Using the far line, even with low oblique imagery, is not recommended since this is the most variable of all the potential locations for measuring GSD. If a location other than the typical GSD reporting location is used, it must be reported in the metadata.

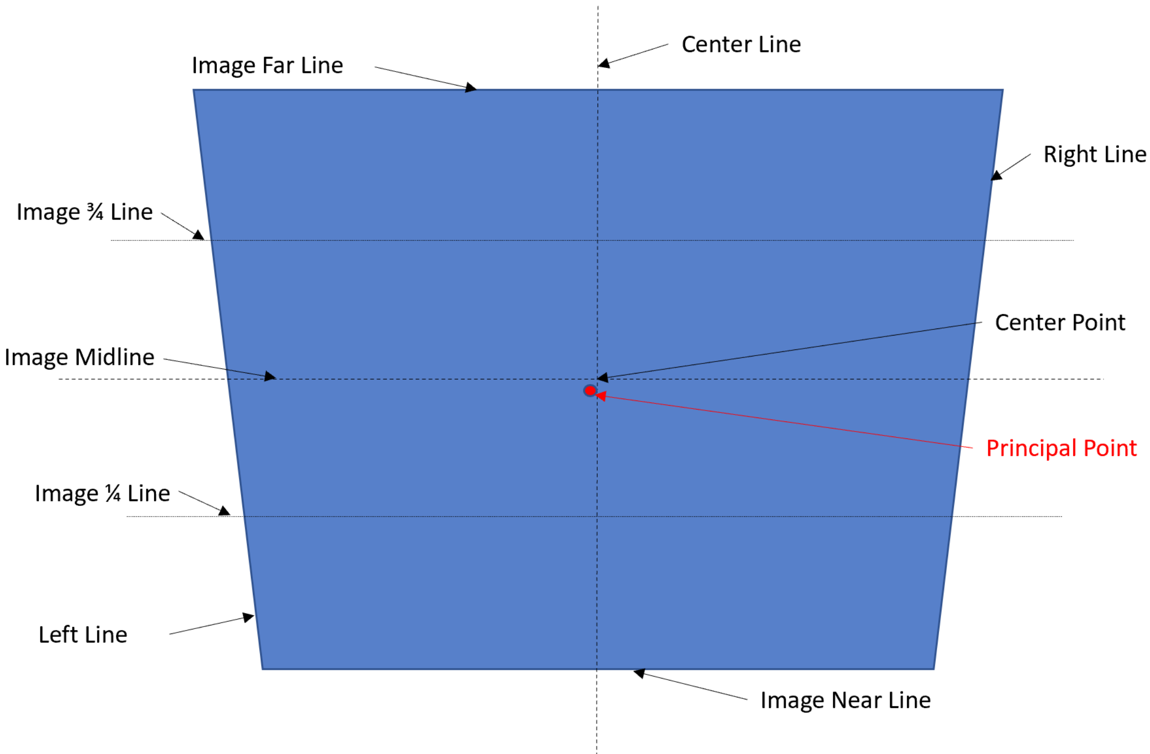

Figure VI.B.2 Geometry of Oblique Image and Definition of Terms (courtesy: David Day)

B.1.4 Reporting Average GSD for Oblique Frame Datasets

When reporting the GSD for oblique frame datasets, the value reported shall be referred to as the average project oblique GSD. It must be calculated as the mean of the individual image frames at either the center point or the principal point, as discussed in section B.1.3. Other positions within the frame may be reported in addition to the center point to correctly report the varying GSD in the frames, but since users should already understand this from the explanations in this section, it is not required. The formula below is included for the removal of doubt, where GSD_i is the measured GSD of oblique image *i* in dataset of size *n*:

$$\text{Average Project Oblique GSD} = \frac{1}{n} * \sum_{i=1}^{n} [\,GSD_i]$$

When reporting the average project oblique GSD, just as with individual perspective images, the following must be included:

- location within the frame: Using the typical GSD reporting location is recommended, but if an alternate location is used, the data producer must report the alternate location
- planned vs. calculated GSD: If calculated, the source of the distances (GNSS Height, AT/bundle, etc.) should be specified
- If ground elevation is used in calculating the GSD, attributions of the elevation model should be documented and reported
- specific oblique view GSDs if not uniform
- minimum, maximum, average, and standard deviations of the average project oblique GSD

The average project oblique GSD must be reported for each of the sensors/oblique views separately. Table VI.B.2 provides examples, including a few atypical systems such as a two-camera system (forward/backward) used to capture four looks via perpendicular flight lines, and a UAS with one camera flying the same flight line twice for forward and backward views.

Table VI.B.3 GSD Reporting Examples for Multiple Scenarios

Typical camera	"The average project oblique GSD for all look angles at the principal point is 10.4 cm generated from bundle adjusted result using an existing height model."
Two-camera system with multiple flights with no principal point	"The average project oblique GSD at the center point is 7.1 cm for N/S views and 7.4 cm for E/W views generated from bundle adjusted result."
UAS-captured imagery with one camera and two separate flights	"The average project oblique GSD at the center point is 1.5 cm for N view and 2 cm for S view generated from computed results."
Camera with two sets of camera types	"The average project oblique GSD at the center point is 9.8 cm for N/S views and 5.2 cm for E/W views generated from bundle adjusted result using the DEM calculated from the dataset."

SECTION C: ACCURACY CONSIDERATIONS FOR OBLIQUE IMAGERY

C.1 Effect of Oblique Geometry

In addition to the common factors that affect vertical imagery accuracy, the accuracy of oblique imagery is also impacted by the following:

1. Degraded image resolution: Because the GSD increases moving farther from the near line of the image in the frame ($A^{\bullet}$ to $B^{\bullet}$ of Figure VI.B.1), image quality is compromised. The deteriorated GSD of the oblique imagery plays a great role in the quality of image matching or manual measurements performed on such imagery. This degradation may be minimized through longer principal distance lenses in the oblique camera design.

2. Degraded image quality: The oblique distance from objects to the camera increases proportionally as the look angle increases (see Table VI.C.1), assuming the project is flown at fixed flying altitude. The longer travel path of the ray entering the camera results in degradation of the image quality due to the increased atmospheric effect. Table VI.C.1 illustrates an increase of 41% (from 2,500.0 to 3,535.5 m) in the path of the midline light ray as it enters the camera. Such an increase becomes 260% at the far line of the oblique image (from 2,500.00 to 6,532.82 m).
3. Exaggerated aerial triangulation errors due to the longer path of travel: Distortion in the ground coverage of a pixel (or GSD) due to the oblique geometry introduces some measurement errors when measurements are performed on the resulting imagery. These inaccuracies influence the quality of the aerial triangulation when refining the exterior orientation parameters of the image, which can result in a less-accurate solution for the camera position and attitudes. Even assuming that the accuracy of the camera attitude angles for the oblique imagery can be determined at a level comparable to the accuracy of the vertical imagery, the extended path of travel by the light ray ($OO^{\bullet\bullet}$ of Figure VI.B.1) results in exaggerated errors compared to those of the nadir imagery. To illustrate this argument, assume that the aerial triangulation solution (or the use of the IMU) resulted in determining the camera attitude (ω, ϕ, **κ**) to an accuracy, $AT_{angle-error}$, of 10 arc seconds. The error on the ground due to these 10 arc seconds, or AT_{error}, at nadir is equivalent to:

   $$\mathrm{AT_{error}} = \text{Flying altitude}\left(OO^{\bullet}\right) \times (\tan\text{(oblique angle)}$$

   While with the oblique:

   $$\mathrm{AT_{error}} = \text{Flying altitude}\left(OO^{\bullet}\right) \times \{\tan\text{(oblique angle} + \mathrm{AT_{angle\text{-}error}}) - \tan\text{(oblique angle)}\}$$

   Table VI.C.1 lists the error values in the mapping products resulting from 10 arc seconds residual error in the camera attitude determination for nadir and 45-degree oblique imagery.
4. Base-to-height ratio and scale variations: Oblique imagery base-to-height ratio compared to vertical imagery is a primary consideration when discussing oblique imagery accuracy. The mapping industry generally considers a 45-degree oblique angle to be ideal for data acquisition, with some variations in the angles between 40 and 50 degrees. Additionally, many typical camera systems use lenses with longer principal distances in their oblique cameras, so that all cameras will produce the same GSD at the mid-oblique regions of the images. For a camera system with oblique cameras mounted at 45-degree look angles, the principal distance for the oblique cameras must be longer than the principal distance of the vertically-mounted camera by a factor of 1.41. This variation in the principal distance among cameras leads to a base-to-height ratio that is 1.41 times worse in the obliques; note that this factor will vary depending on the camera's principal distance ratios and/or the oblique look angle.

Table VI.C.1 Error in Mapping Product Resulting from Error in Camera Attitude Determination (courtesy: Dr. Qassim Abdullah)

Flying Altitude AGL (m)	Mid-Oblique Distance (m)	Horizontal Error in X or Y at Nadir (m)	Horizontal Error in X or Y at Oblique (m)	% Error Increase in Oblique Image
500.0	707.1	0.024	0.048	100%
1000.0	1414.2	0.048	0.097	100%
1500.0	2121.3	0.073	0.145	100%
2000.0	2828.4	0.097	0.194	100%
2500.0	3535.5	0.121	0.242	100%
3000.0	4242.6	0.145	0.291	100%
3500.0	4949.7	0.170	0.339	100%

C.2 Estimating Horizontal and Vertical Accuracy for Oblique Imagery

Because of the unique nature of oblique data products, positional accuracy must be given special consideration. When deriving oblique data accuracy, the vertical positional accuracy is the best starting point. Consult Section 7 of the ASPRS Positional Accuracy Standards for Digital Geospatial Data, Edition 2, Version 2.0 (2024), referred to here as the ASPRS Standards.

All statements from Section 7 of the ASPRS Standards regarding the quality and distribution of the ground control points and checkpoints—as well as those for the GNSS/INS data used in the aerial triangulation—are applicable to oblique aerial triangulation.

The most direct way to model accuracy and error propagation in oblique imagery is by using the well-known accuracy and error modeling for traditional vertical imagery based on the collinearity conditions. For this type of accuracy modeling, the oblique image is used to generate a virtual vertical image at the same location (i.e., the oblique and vertical images share the same perspective center) with the same principal distance. The collinearity equations are then used to establish a mathematical relationship between corresponding oblique and vertical image coordinates. Finally, these coordinates are used for the error propagation and accuracy predictions. When predicting accuracy using vertical images, the established formulas (as per Appendix VI.A) should be used. Figure VI.C.1 shows the oblique and virtual vertical images for a stereo-pair. This figure shows how the mathematical relationship between XY-image coordinates in the oblique and virtual vertical images is derived. Note: The presented equations assume that the object space is planar and encompasses the XY-axes of the ground coordinate systems (i.e., $h = 0$).

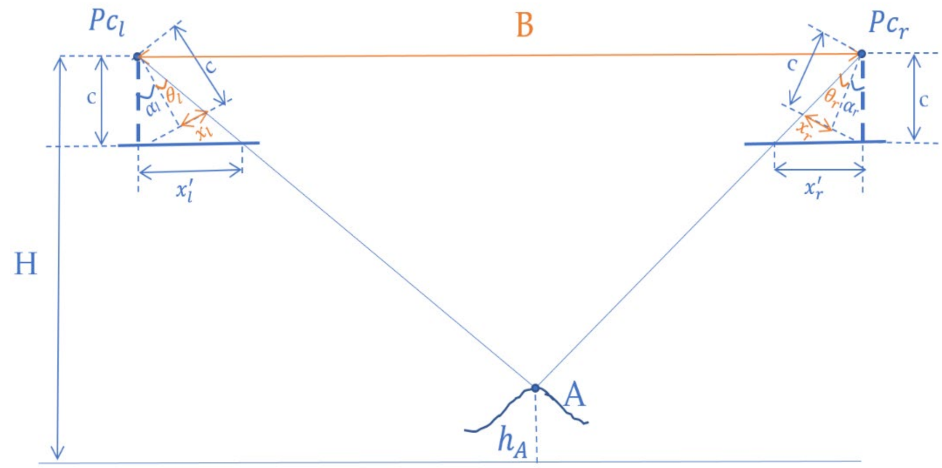

Figure VI.C.1 Stereo-pair geometry for oblique and virtual vertical image stereo-pair (courtesy: Dr. Ayman Habib)

C.2.1 X-Accuracy

Using the collinearity equations, one can derive a mathematical relationship between the X-image coordinates of the given point in the oblique image and the corresponding virtual vertical image, as shown in Equation 1:

$$x' = c\,\frac{\cos\alpha\ x + \sin\alpha\ c}{\cos\alpha\ c - \sin\alpha\ x} \qquad 1$$

Using the accuracy formulas for vertical images, the X-ground coordinate accuracy can be derived as in Equation 2, where the highlighted term in red represents the deterioration in the accuracy as a result of using an oblique image:

$$\sigma_X = \frac{H}{c}\,\sigma_{x'y'} = \frac{\mathbf{c^2}}{(\mathbf{\cos\alpha\ c - \sin\alpha\ x})^2}\,\frac{H}{c}\,\sigma_{xy} \qquad 2$$

As an example, for an oblique image captured by a camera with the following specifications: c = 100 mm, $\alpha = 45$ degrees, camera format = 100 MP, pixel size = 0.00376 mm; Figure VI.C.2 illustrates the percentage of X-accuracy deterioration over the image format when compared to a vertical image; i.e., $\left(\frac{c^2}{(\cos\alpha\ c - \sin\alpha\ x)^2} - 1\right) * 100$.

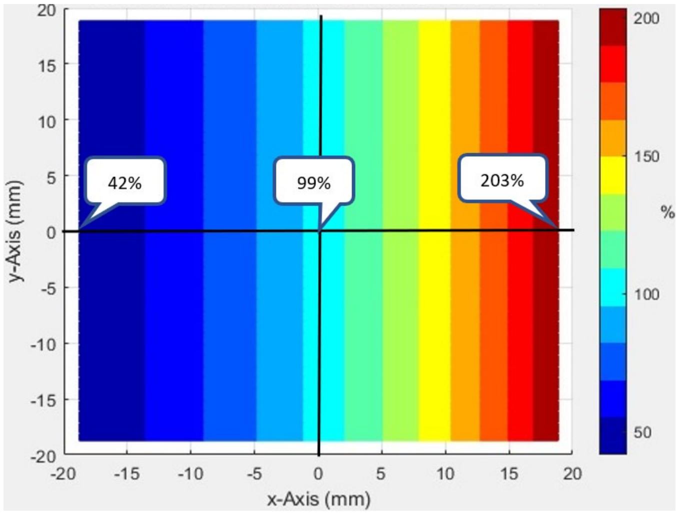

Figure VI.C.2 Top view of X-accuracy deterioration percentage over the image format (courtesy: Dr. Ayman Habib)

C.2.2 Y-Accuracy

In a similar fashion, the Y-ground coordinate accuracy can be derived as in Equations 3 and 4, where the highlighted term in red represents the deterioration in the accuracy as a result of using an oblique image.

$$y' = \frac{c\,y}{\cos\alpha\ c - \sin\alpha\ x} \tag{3}$$

$$\sigma_Y = \frac{H}{c}\,\sigma_{x'y'} = \frac{c\sqrt{\sin^2\alpha\, y^2 + (\cos\alpha\ c - \sin\alpha\ x)^2}}{(\cos\alpha\ c - \sin\alpha\ x)^2}\,\frac{H}{c}\,\sigma_{xy} \tag{4}$$

As an example, for an oblique image captured by a camera with the following specifications: c = 100 mm, $\alpha = 45$ degrees, camera format = 100 MP, pixel size = 0.00376 mm; Figure VI.C.3 shows the deterioration percentage of the Y-accuracy over the image format when compared to vertical image; i.e.,

$$\left(\frac{c\sqrt{\sin^2\alpha\, y^2 + (\cos\alpha\ c - \sin\alpha\ x)^2}}{(\cos\alpha\ c - \sin\alpha\ x)^2} - 1\right) * 100.$$

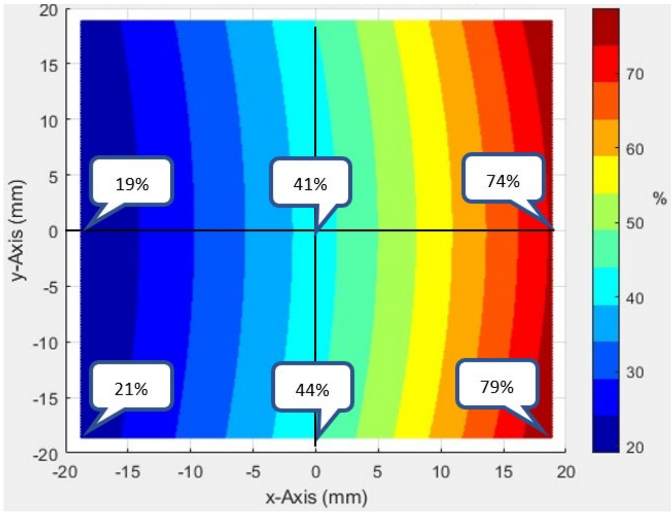

Figure VI.C.3 Top view of the Y-accuracy deterioration percentage over the image format (courtesy: Dr. Ayman Habib)

C.2.3 Horizontal Accuracy

From equations 2 and 4, the horizontal accuracy for oblique imagery can be computed via the following equation, assuming that there are no biases in the estimated values so that the σ_X and σ_Y values approach $RMSE_X$ and $RMSE_Y$, respectively:

$$RMSE_H = \sqrt{{\sigma_X}^2 + {\sigma_Y}^2}$$

C.2.4 Z-Accuracy (Vertical Accuracy)

Using the collinearity equations for an oblique stereo image pair as outlined in Figure VI.C.1, the x-parallax ($p_{x'}$) for the virtual vertical stereo-pair can be established as per Equation 5. The Z-accuracy of an object point can be derived according to Equation 6, where the highlighted term in red represents the deterioration in the accuracy as a result of using oblique images for Z-coordinate derivation.

$$p_{x'} = x_l' - x_r' = c\,\frac{\cos\alpha_l\ \ x_l + \sin\alpha_l\ \ c}{\cos\alpha_l\ \ c - \sin\alpha_l\ \ x_l} - c\,\frac{\cos\alpha_r\ \ x_r + \sin\alpha_r\ \ c}{\cos\alpha_r\ \ c - \sin\alpha_r\ \ x_r} \tag{5}$$

$$\sigma_Z = \frac{H}{B}\frac{H}{c}\sigma_{p_{x'}} = \frac{\boldsymbol{c}^{\,\mathbf{2}}}{\sqrt{\mathbf{2}}}\sqrt{\frac{\mathbf{1}}{(\cos\boldsymbol{\alpha_l}\ \boldsymbol{c} - \sin\boldsymbol{\alpha_l}\ \boldsymbol{x_l})^{\mathbf{4}}} + \frac{\mathbf{1}}{(\cos\boldsymbol{\alpha_r}\ \boldsymbol{c} - \sin\boldsymbol{\alpha_r}\ \boldsymbol{x_r})^{\mathbf{4}}}}\ \sqrt{2}\frac{H}{B}\frac{H}{c}\,\sigma_{xy} \tag{6}$$

To illustrate the deterioration percentage in the vertical accuracy, we consider two examples where the height is derived using either 1) left and right oblique images or 2) left oblique and right vertical images. These stereo-images are captured from two neighboring flight lines. One should note that these examples would also be valid for 1) forward and backward oblique images or 2) forward oblique and vertical images. These stereo-images are captured from the same flight line.

Example 1: Left and right oblique stereo-pair

For a camera and imaging configuration with the following specifications: c = 100 mm, $\alpha_l = 45$ degrees, $\alpha_r = -45$ degrees, flying height = 1000 m, airbase = 2000 m, camera format = 100 MP, pixel size = 0.00376 mm; Figure VI.C.4 represents the ground coverage of the left and right oblique images including the area of overlap. Figure VI.C.5 illustrates a plot of the Z-accuracy deterioration percentage for the overlap area between stereo oblique images; i.e.,

$$\frac{\left(c^2\sqrt{\frac{1}{(\cos\alpha_l\ c-\sin\alpha_l\ x_l)^4}+\frac{1}{(\cos\alpha_r\ c-\sin\alpha_r\ x_r)^4}}-1\right)}{\sqrt{2}}*100.$$

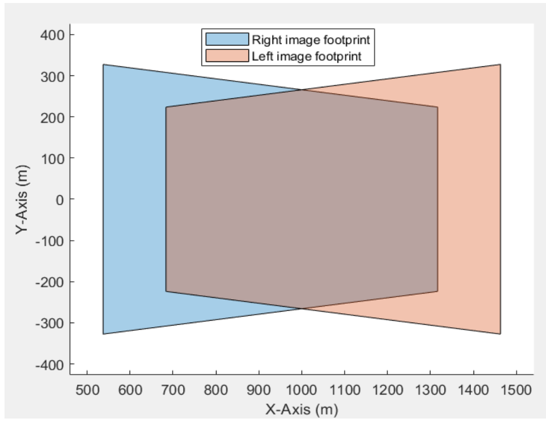

Figure VI.C.4 Ground coverage of stereo oblique images from two neighboring flight lines (courtesy: Dr. Ayman Habib)

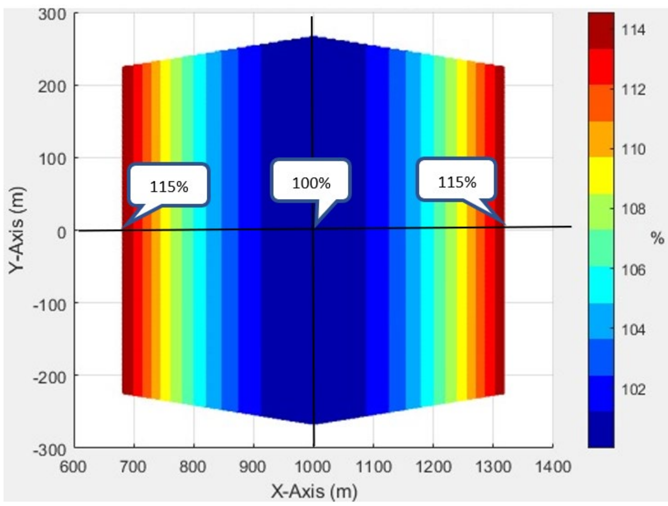

Figure VI.C.5 Top view of the Z-accuracy deterioration percentage for stereo oblique images from two neighboring flight lines (courtesy: Dr. Ayman Habib)

Example 2: Left oblique and right vertical stereo-pair

For a camera and imaging configuration with the following specifications: c = 100 mm, $\alpha_l = 45$ degrees, $\alpha_r = 0$ degrees (vertical image), flying height = 1000 m, airbase = 1000 m, camera format = 100 MP, pixel size = 0.00376 mm; Figure VI.C.6 represents the ground coverage of the left oblique and right vertical images, including the area of overlap. Figure VI.C.7 illustrates a plot of the Z-accuracy deterioration percentage in the overlap area between the oblique and nadir images; i.e.,

$$\frac{\left(c^2\sqrt{\frac{1}{(\cos\alpha_l\ c-\sin\alpha_l\ x_l)^4}+\frac{1}{(\cos\alpha_r\ c-\sin\alpha_r\ x_r)^4}}-1\right)}{\sqrt{2}}*100.$$

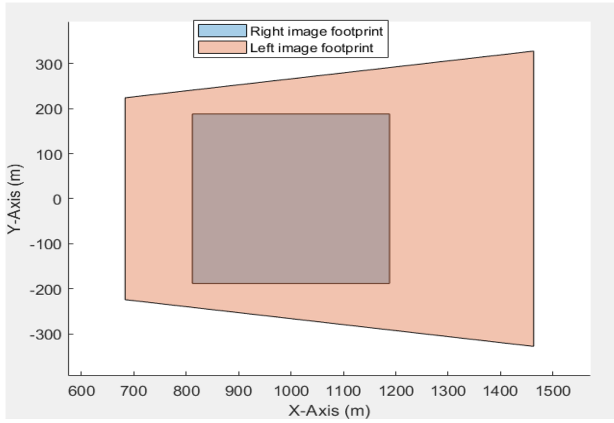

Figure VI.C.6 Ground coverage of a stereo-pair formed by left oblique and right vertical images from two neighboring flight lines (courtesy: Dr. Ayman Habib)

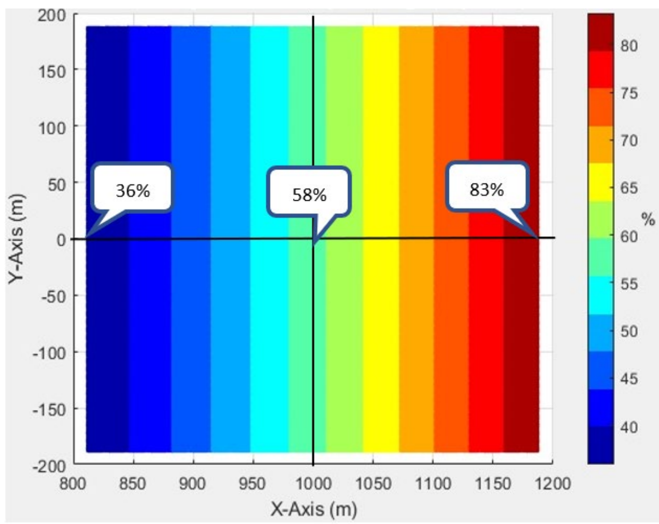

Figure VI.C.7 Top view of the Z-accuracy deterioration percentage for a stereo-pair formed by oblique and nadir images from two neighboring flight lines (courtesy: Dr. Ayman Habib)

Note: Equations 2, 4, and 6 are valid for vertical images, where the off-nadir angle (α) is simply zero. In other words, using an off-nadir angle of zero in these equations would result in the same formulas for vertical images, as per Appendix VI.A.

C.2.5 3D Accuracy

From equations 2, 4, and 6, the three-dimensional accuracy for oblique imagery can be computed as follows, assuming that there are no biases in the estimated values so that $\sigma_{X,}$ σ_Y, and σ_Z values approach $RMSE_{X,}$ $RMSE_{X,}$ and $RMSE_Z$, respectively:

$$RMSE_{3D} = \sqrt{{\sigma_X}^2 + {\sigma_Y}^2 + {\sigma_Z}^2}$$

C.2.6 Recommended Strategy for Estimating the Accuracy of Oblique Imagery Products

The discussion above presents the theoretical error modeling for the photogrammetric geometry. However, there are additional errors introduced during product generation using oblique imagery. Oblique geometry results in non-homogeneous overall accuracy in the product, due to the additional factors which affect oblique imagery accuracy.

Because the accuracy of products derived from vertical imagery is well-modeled, this Addendum recommends measuring the accuracy of products generated from oblique imagery by combining the oblique and vertical imagery into one aerial triangulation adjustment, then comparing the oblique image accuracy to the accuracy of products derived from vertical imagery of the same flight mission. The following sections provide further details on strategies for estimating the accuracy of products derived from oblique imagery.

C.2.7 Estimating Oblique Degradation Factor

The most significant sources of accuracy degradation in products derived from oblique imagery are geometrical modeling error and scale variation factors. Geometrical modeling error is derived from the photogrammetric stereoscopic measurement—this is illustrated in the examples given in Sections C.2.1 through C.2.4. The size of the scale variation factor will depend on the sensor size and the FOV; as GSD increases toward the edge of the image, so do the errors. This mainly impacts the accuracy in the far-oblique region; for mid-oblique, this factor should not be used in the accuracy estimation.

Aside from the geometric model imperfection, additional factors may cause deviation from the standard geometric model, such as mechanical tolerances during the production of an oblique camera system, deviations from the ideal square root 2 factor in terms of principal distance (c) ratios, input quality regarding the sensitivity of navigational data (IMU), and atmospheric conditions.

C.2.8 Recommended Expression of Oblique Product Accuracy

It is generally safe to assume that products derived from oblique imagery are less accurate than those derived from vertical imagery. A degradation factor of 2–3 can be used to estimate the accuracy of oblique products compared to the accuracy of products derived from vertical imagery with comparable flying parameters.

Accuracy measures may vary widely depending on operational conditions, system design and configuration, camera, lens quality, etc. Therefore, this value should only be used for initial estimation and project design. The actual accuracy of products derived from oblique imagery can only be assessed and reported using independent checkpoints according to the guidelines outlined in this Addendum and these Standards.

SECTION D: RADIOMETRY

Oblique image capture utilizes a tilted sensor to capture objects in a scene. The atmosphere between the sensor and the scene will generally increase as a function of the look angle. The variability in atmospheric obstruction creates radiometric effects seen in the edge of the field, which reduce contrast and may introduce color shift. By their nature, oblique images are more likely to be affected adversely by looking more closely into or away from solar effects than vertical images. This usually introduces glint and negatively affects the ability to resolve content at the extremes of a sensor's dynamic range. These characteristics may be partially accounted for in post-capture production, but these should be considered during planning and capture. Best practices for the radiometric correction of digital imagery can be found in the USGS Digital Imagery Guideline[1]. Additionally, radiometric calibration in a lab setting can be important in the production of products and should be considered for both typical oblique camera systems as well as non-typical systems that may be flown on platforms such as UAS.

SECTION E: OBLIQUE IMAGING SYSTEM CONFIGURATION

E.1 Look Angle and Image Eccentricities

The look angle is the angle between a line from the camera station to a particular element in a scene and the nadir axis. Due to the foreshortening effect in oblique imagery, a typical camera system will have the oblique cameras mounted at look angles between 40 and 50 degrees. While this range is generally recommended, there are valid reasons to use other look angles, especially when the camera system is paired with other sensor types and coverage matching is desired. Data producers should ensure that the project's required accuracies can be attained and that the methodologies for achieving and verifying those accuracies are documented.

For the 3D modeling of specific structures or small areas, data producers should follow their software recommendations while generating products. For such modeling, images using various look angles and distances from the point of interest are recommended when processing the imagery.

E.2 Image Motion Compensation

It is well-known that image motion from aircraft movement, primarily in the direction of flight (forward motion), is the largest factor when considering image resolution and blurring of aerial remote imaging. For oblique imagery, image motion compensation must additionally account for the scale variance within the oblique perspective image. Angular motion during image capture adds to this problem by causing non-forward motion that affects the oblique cameras in various directions and amounts.

At the minimum, data producers must ensure that motion blur is reduced so that the amount of blur is less than the average project oblique GSD (section B.1.4),especially for the processes that the data producer can control. Blur reduction is essential for creating high quality photogrammetric image derivatives such as DSM, DSM orthophotos, and 3D models.

There are two types of compensation technologies: those controlled during imagery acquisition, and those controlled post-acquisition. This section of the Addendum is intended to inform data producers and data users about the different control methods available so that they can understand the benefits and limitations each system offers according to their project needs.

The data producer must disclose the methods used to reduce blur in a dataset as part of the metadata or notes. Additionally, the data producer is responsible for ensuring that data is collected using the appropriate aircraft speed, scene lighting, and exposure settings for blur reduction, as per camera

[1] Digital Imagery Guideline. https://www.usgs.gov/media/images/spatial-resolution-digital-imagery-guideline

manufacturer recommendations. It is crucial that these parameter settings are followed, especially when these methods are the only means for blur minimization in a system.

Camera systems may adjust for aircraft motion and angular movement of the camera system using a combination of one or more of the methods discussed in Table VI.E.1. All the methods mentioned in Table VI.E.1 benefit from using reasonably fast exposure times. There is a strong correlation between the exposure time used and the vulnerability of a camera system to angular motion blur.

Note that gyro-stabilized mounting systems do not inherently compensate for angular motion blur. While these systems can provide extra stability, they would need to move a large aerial camera in fractions of seconds in order to reduce angular motion blur in the captured image.

Table VI.E.1 Common Methods for Image Motion Compensation

Method	Pros	Cons
External Factors (controlled by the data acquisition team): Blur reduction via fast exposure times, proper aircraft/UAS speed, and proper ground lighting control.	Simple to implement. May be an essential component for the proper application of the other methods. For smaller systems like the ones typically used in UASs, this is often the most reasonable approach due to the weight and power considerations for other methods.	Depending on image scale and flight parameters, full motion compensation may not be possible. Could result in longer collection times due to reduced speed. Due to the dependency on exposure time, noise in the image may be increased to unacceptable levels.
Physical/Mechanical Compensation (part of camera design): Moving parts of the sensor at the time of exposure to match the aircraft's speed.	Can fully compensate motion in flight direction. Can be used with sub-pixel accuracy.	Does not compensate for angular motion and scale differences. Requires mechanical moving parts.
Digital Compensation/Time Delayed Integration (TDI) (part of camera design): Typically used with CCD cameras to move the charge with the aircraft's motion.	Can fully compensate motion in flight direction.	Limited to CCD camera systems. Cannot be used with sub-pixel accuracy. Does not compensate for angular motion and scale differences.
Software Compensation (part of system design): The use of GNSS/IMU data to remove angular motion blur. This method is adaptable to oblique image scale variations. It relies on a precise GNSS/IMU system, and the results are available after post-processing.	Can compensate for motion blur in all directions. Can compensate for scale differences in oblique imagery. Can be used with sub-pixel accuracy. No mechanical moving parts.	Requires a good quality IMU, like those typically installed in large aerial cameras. Requires a terrain model for best results of the compensation algorithm. Could be an expensive option for certain projects.

It is the responsibility of the data user and data producer to select blur reduction methods appropriate to the project type and end use. The two parties should also set metrics for determining and measuring blur, as well as permissible amounts of blur in the images and permissible blurred image counts within a dataset.

Resolution is an imaging system's ability to distinguish object detail. Factors such as lens optics, sensor quality, and atmospherics affect the resolution. The example in Figure VI.E.1 highlights the large effect of blur (forward and angular motion) on image sharpness and clarity, even with an identical GSD.

Figure VI.E.1 Image Motion Compensation Example - Raw image (top) Blur-Free Image (bottom) (courtesy: Vexcel Imaging)

SECTION F: MISSION PLANNING DATA PROCESS

F.1 Coverage

The data user must specify the number of oblique views for all objects/locations within the delivery area of interest (AOI). To meet this specification, the data producer must plan the acquisition with the correct number of frames, lines, passes, etc. For typical camera systems, this will mean flight lines outside the AOI boundaries for complete coverage of the AOI with the side oblique views, as well as imagery collection past the AOI in line for forward and backward views.

F.2 Obstructions

The data producer and data user must agree how to handle building obstructions in dense urban collections, whether by dense line spacing or by acceptable obstruction criteria in certain areas.

For 3D modeling specific structures or small areas, especially with UAS platforms, the software manufacturer's specifications for overlap, altitude/angle variation, flight plan design, etc. must be followed.

F.3 Standard Oblique Project Planning

For a standard oblique project where four oblique views are captured in each cardinal direction plus nadir imagery, these guidelines must be followed in order to generate high-quality oblique imagery datasets:

- There must be full oblique overlap throughout the project at the target GSD.
- Forward overlap for all four cardinal directions should be the minimum necessary to ensure there are no gaps at the near line between overlapping frames of each individual oblique look angle throughout the project.
- Side overlap must be such that there is overlap of the side oblique images between flight lines. For example: line 1 left oblique and line 2 left oblique must have overlap, as shown in Figure VI.F.2.
- During planning, the along track overlap between frames should be measured between the near and far lines for forward and backward obliques, and the left and right lines in lateral/side obliques, as shown in Figures VI.F.1 and VI.F.2.

Additional explanations for the along-track overlap illustrations are explained below. For simplicity in these diagrams, the footprints are estimated by altitude and camera specifications without the effects of terrain considered. Terrain variation will impact final overlap measurements on actual oblique images. The data producer should explain the methodology used to calculate overlap on the final obliques.

- Forward oblique frames: The overlap is measured from the initial image far line to the subsequent image near line, as shown in Figure VI.F.1.
- Backward oblique frames: The overlap is measured from the initial image near line to the subsequent image far line, as shown in Figure VI.F.1.
- Left oblique frames: The overlap is measured at the midline from the initial image right line to the subsequent image left line, as shown in Figure VI.F.2.
- Right oblique frames: The overlap is measured at the midline from the initial image left line to the subsequent image right line, as shown in Figure VI.F.2.

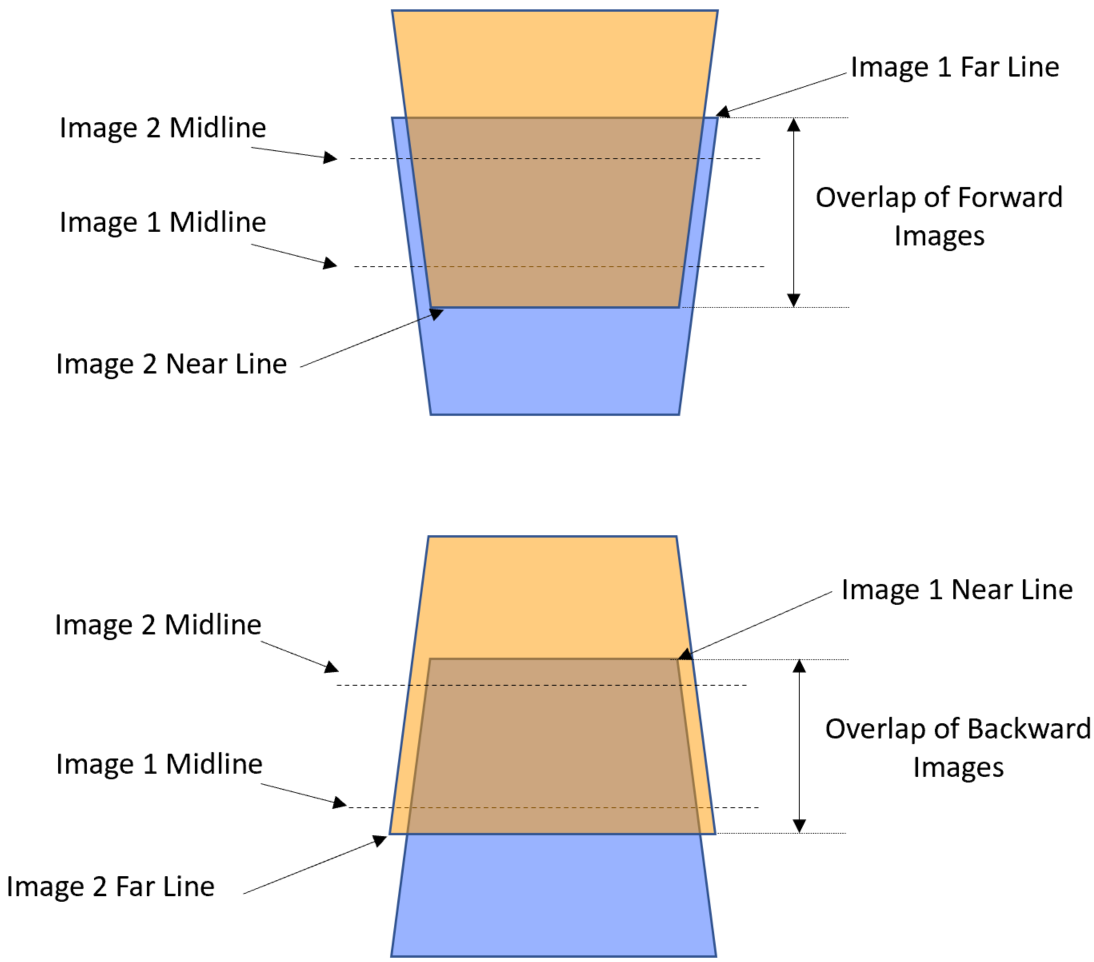

Figure VI.F.1 Forward and Backward Oblique Image Set Overlap Definitions (courtesy: David Day)

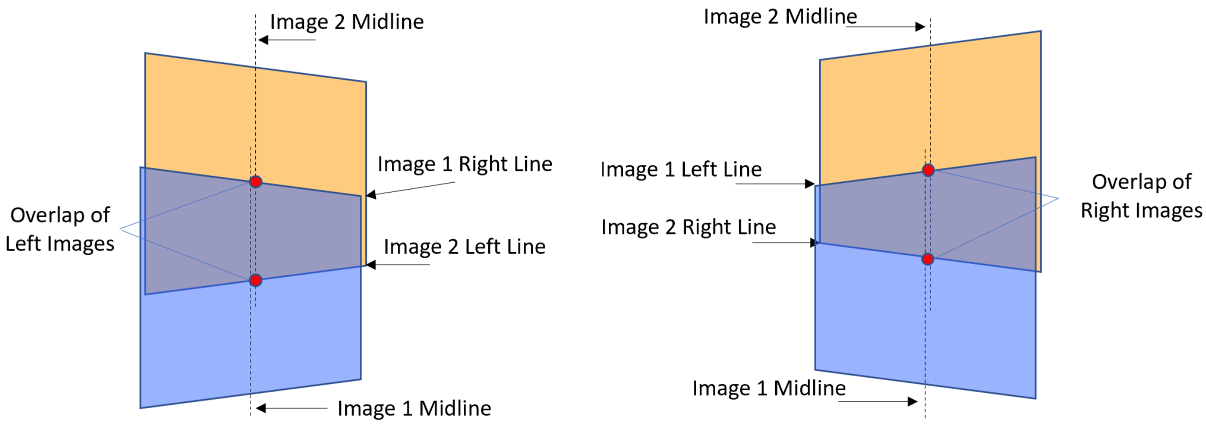

Figure VI.F.2 Side Oblique Image Set Overlap Definitions (courtesy: David Day)

During planning, the side overlap should be measured as described in Figure VI.F.3. For simplicity, these footprints are estimated without the effects of terrain, and can be applied to typical cameras or a camera with any number of oblique views.

Notes on side overlap illustrations:

- Forward and backward oblique side overlap is measured from the right line of the first image to the left line of the adjacent image along the midline of the images.
- Lateral oblique side overlap is measured from the image far line of the first image to the near line of the adjacent image along the center line of the images.

It should be noted that actual individual image footprints will vary from the flight plan due to terrain variation across each flight line. Data producers should specify the methodology used to measure the overlap on the captured oblique images.

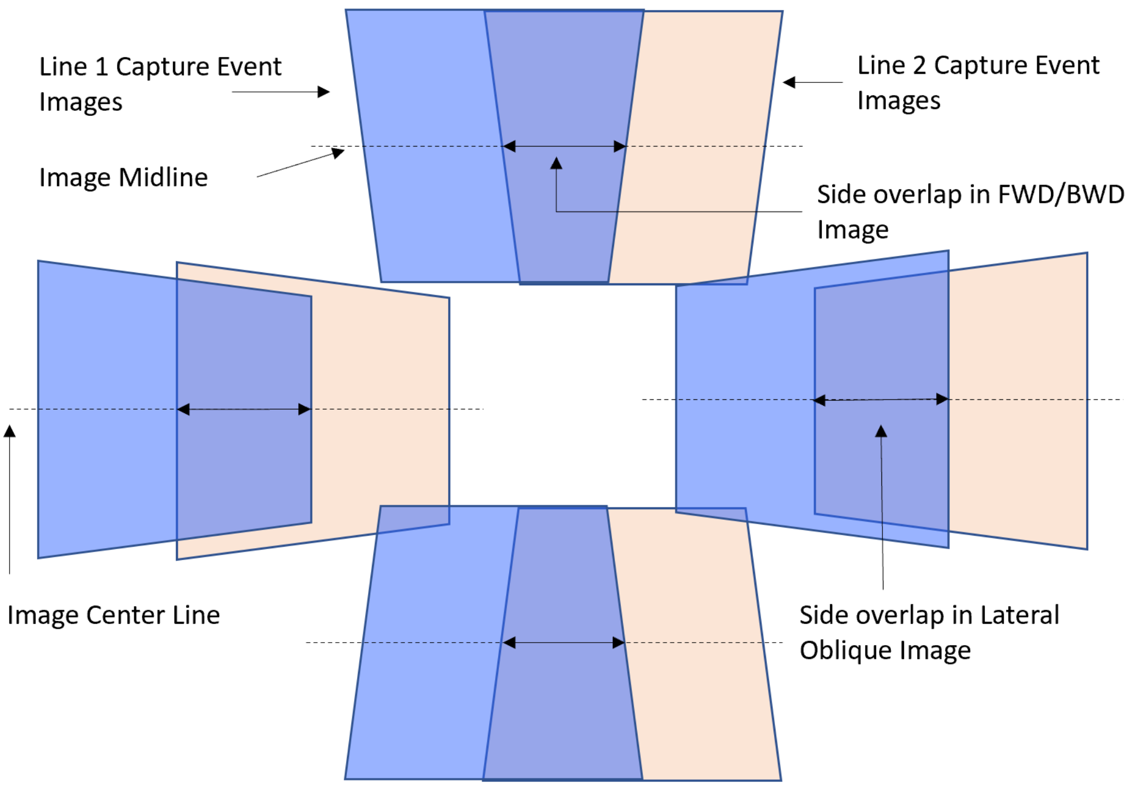

Figure VI.F.3 Full Oblique Frame Set for Typical Oblique Camera (courtesy: David Day)

F.4 Height Models and Planning

The effect of terrain on coverage and GSD of a nadir image is well-known, but, in oblique images, this effect is magnified by the trapezoidal shape of the image footprint when the terrain is applied. When determining the GSD and coverage for an oblique project, there are two commonly used methods. The data producer must be clear on which method has been followed. The first and most accurate method to estimate the coverage footprint of oblique images is to use an accurate height model. Line/image spacing will vary within a project depending on topography, thus affecting the overall images needed in a collection. However, the technological ability and manual effort necessary to perform this type of planning is usually not realistic for most projects.

The second method is to plan without height models by assuming flat earth, and then use very high overlap and sidelap in order to assure complete coverage of the AOI with obliques. Height models should still be used for planning nadir collection. This will result in complete overlap of all oblique images throughout the dataset, along with higher oblique overlap percentages.

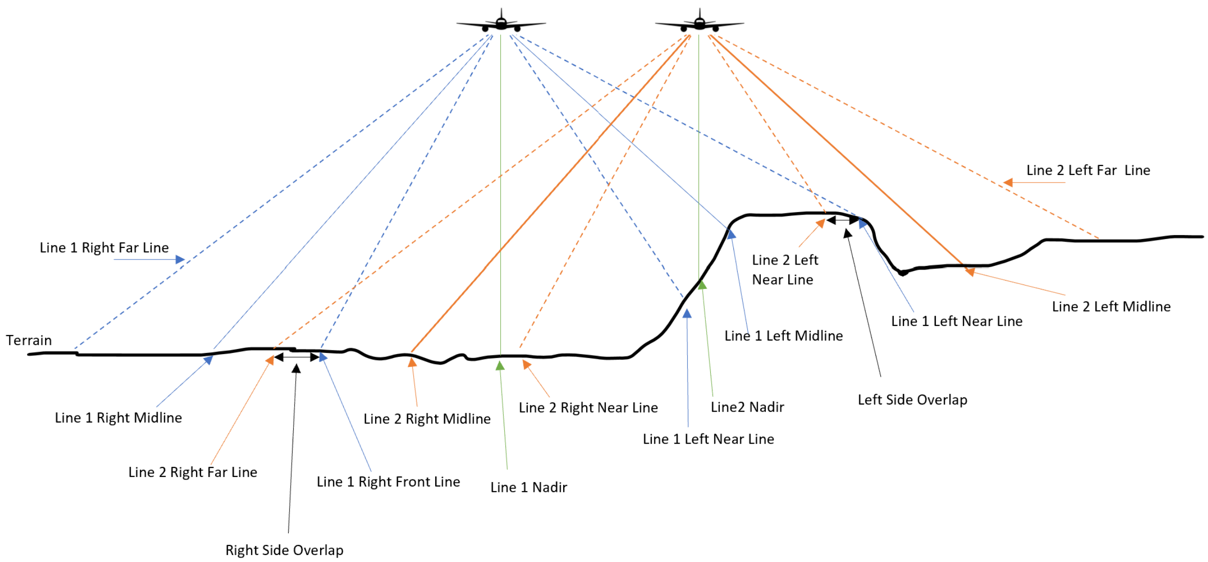

Figure VI.F.4 Illustration of Terrain Effects on Flight Planning for Oblique Imagery Collections (courtesy: David Day)

F.5 Flight Plan Reporting

The flight planning report must include a description of and discuss the following, in the level of detail agreed upon by the data user and data producer:

- Height model: The type of height model used in planning (if any) and its application in the planning. Specifically, if it was used for estimations of the GSD, overlap, and side overlap, and for which camera(s), if any.
- GSD: Report of the estimated average GSD and min/max for the project for at least one selected camera (nadir or oblique).
- Standard deviations (optional)
- Estimations for all cameras (optional)
- Tolerances and variances: Should be acceptable to both parties regarding products generated from the dataset.
- Overlap: Report of the estimated average along track overlap and min/max for the project for at least one selected camera (nadir or oblique).
- Side overlap: Report of the average distance on the ground and min/max between flight lines, expressed as a percentage for a specific camera (nadir or oblique).

Additional computed parameters in the flight planning may need to be reported according to Addendum I, Section A of these Standards.

SECTION G: DATA PROCESSING AND PRODUCTION

The production process for oblique imagery follows the typical photogrammetric workflow traditionally used for vertical imagery. Figure VI.G.1 illustrates the typical photogrammetric process.

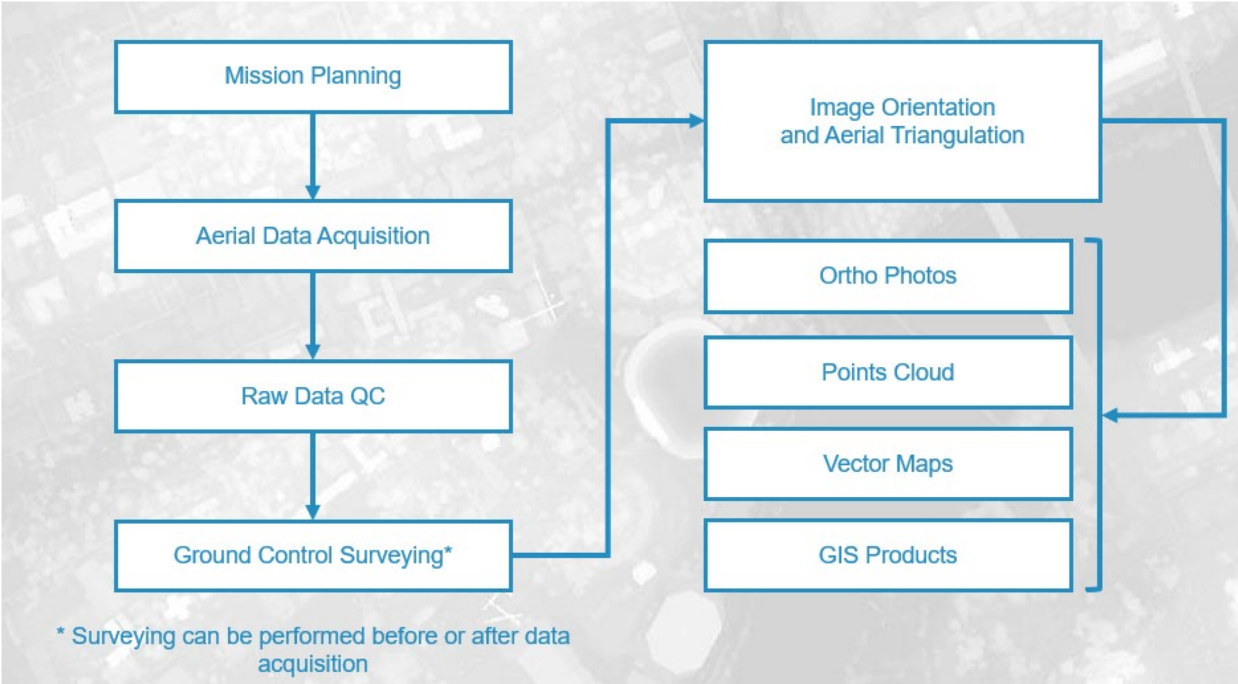

Figure VI.G.1 Photogrammetric Data Processing Workflow (courtesy: Dr. Qassim Abdullah)

G.1 Aerial Triangulation

Aerial triangulation with bundle adjustment or other post-production techniques, such as Structure from Motion (SfM), should be used to refine the positions and orientation of imagery of the project. Aerial triangulation should be performed using the GNSS-derived image center, ground control points (if high accuracy is anticipated from the products), and tie points generated using both oblique and nadir imagery. Additionally, the adjustment algorithms must properly correct for atmospheric distortion, earth curvature, and radial symmetric lens distortion for all imagery in the project. Details regarding the software, parameter settings, camera self-calibration, and general workflow must be reported in the metadata or project notes.

G.2 Data Quality Assessment

Once data is captured and processed, oblique data should undergo a thorough quality control process to detect and correct errors. The quality control process may involve visual inspection, statistical analysis of accuracy, or comparison with ground truth data. To assess positional accuracy, data users and data producers should follow the best practices and guidelines outlined in these Standards.

G.3 Software Display and Measurement

Oblique imagery products should be evaluated using a true oblique viewer that is capable of establishing the oblique geometry. The display and measurement software for oblique aerial imagery should have the minimum standard navigation features. Minimum additional features that should be included are:

- Date and time of the imagery associated with the pixel selected

- The GSD of the frame as calculated following the guidelines in this Addendum, or preferably, the GSD of the specific pixel
- The cardinal direction of the oblique view
- Full metadata as described in Section G.4, which should be available for each oblique view and the nadir image (or orthomosaic dataset)
- Measurement tools, specifically for distance, height, or area, with a description of the calculation method

G.4 Metadata

G.4.1 Standard Metadata Reporting

At minimum, the data producer must include the items listed below in any dataset or frame metadata. The user of the oblique dataset must require the metadata be given in a standard geospatial data format (ISO, FDGC, etc.). The data producer must incorporate the items listed into the appropriate section(s), as defined by the metadata format. The following list assumes that camera specifications for each type of camera in the camera system are included in the metadata format. If this is not the case, the minimum camera information should be listed as well:

- Average project oblique GSD (reported as specified above) or GSD of the specific image
- Average or specific lateral and forward/backward overlap by percentage of nadir, each look angle, and absolute distance between lateral capture events
- Average side overlap between lines in each look angle
- Average or specific image eccentricities or look angle for each oblique image, reported in degrees from nadir
- Lens field of view, reported as horizontal degrees by vertical degrees
- Average look angle deviation from true north in degrees
- Format and compression of the oblique images if different from the nadir
- Camera description for each camera (if not included elsewhere):
    - Principal distance (c) in millimeters
    - Frame size in horizontal pixel by vertical pixels
    - Pixel size in micrometers
    - Principal point (if known)
    - Reference system for principal point (for example, X right and Y Up, X Up and Y Left, etc.)
    - Distortion parameters (if known), with details about calculation method (photogrammetric vs. computer vision) and additive distortion vs. removal of distortion
    - If the oblique images have been rotated after collection (e.g., horizon-up) then this rotation should be noted in the metadata and delivery notes. In addition, the adjustments or lack of adjustment to these values must be documented accordingly, as this rotation will affect the principal point coordinates, camera distortion values, and the exterior orientation rotation values

G.5 Exterior Orientation File Contents

In the event that the agreed deliverables of an oblique aerial imagery collection include perspective images (as defined above) with an exterior orientation (EO) file, the data producer must include the following information in accordance with Addendum I, Section A.2 of the ASPRS Standards:

- The rotation sequence of the exterior orientation angles for each oblique aerial image (ω, ϕ, κ)
- The rotation order (for example, right- or left-handed)
- Coordinate system definition, horizontal and vertical datum, and units of measure
- Off-nadir angle for each oblique camera
- Notes on the determination of the rotation around the center point or principal point of the oblique image
- Notes on the orientation of the camera system in the aircraft or UAS

G.6 Reporting Notes

In addition to the EO files for discrete datasets, reporting notes should include system calibration reports for each oblique camera (refer to Addendum I, Section A.2 of the ASPRS Standards for additional reporting requirements) and the following:

- Frame size: Both physical and in pixels
- Pixel size
- Off-nadir rotation angle
- The rotation angle from the nadir camera (in a typical camera system)
- Principal point
- Lens principal distance

APPENDIX VI.A VERTICAL IMAGE ACCURACY

VI.A.1 Planimetric Accuracy

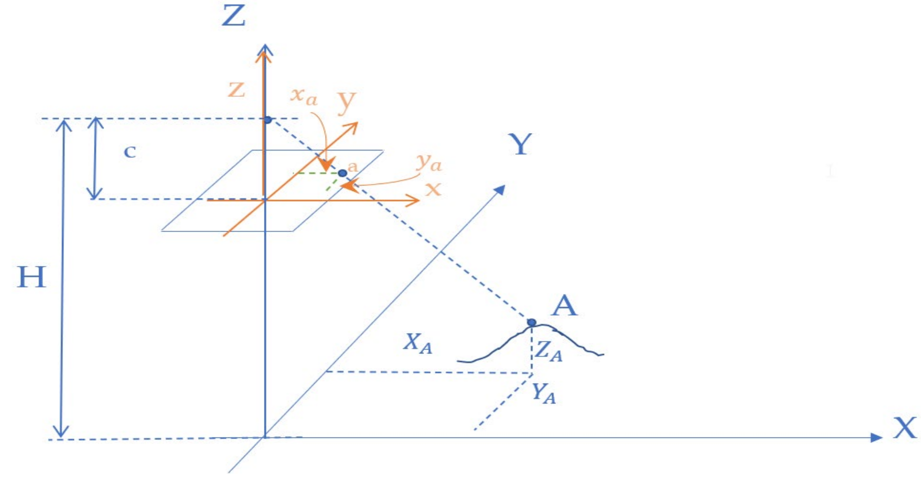

Figure VI.AppendixA.1 Vertical Image Geometry (courtesy: Dr. Ayman Habib)

$$X_A = x_a \frac{(H - Z_A)}{c} = x_a \frac{(H - h_A)}{c}$$

$$Y_A = y_a \frac{(H - Z_A)}{c} = y_a \frac{(H - h_A)}{c}$$

Accuracy estimate based on the law of error propagation:

$$\sigma_X = \frac{(H - h)}{c}\, \sigma_{xy}$$

$$\sigma_Y = \frac{(H - h)}{c}\, \sigma_{xy}$$

Where:

- σ_{xy} = Image coordinate measurement accuracy
- σ_X = Accuracy of derived ground coordinates in X-direction
- σ_Y = Accuracy of derived ground coordinates in Y-direction

VI.A.2 Vertical Accuracy

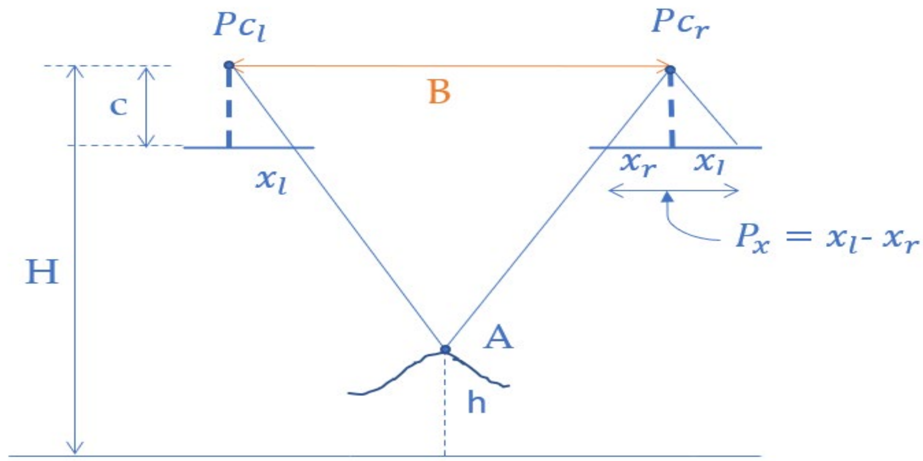

Figure VI.AppendixA.2 Stereo-pair geometry and derivation of vertical accuracy (courtesy: Dr. Ayman Habib)

$$\sigma_Z = \sqrt{2}\ \frac{(H-h)}{B}\frac{(H-h)}{c}\ \sigma_{xy}$$

Assumptions:

- Vertical image
- Only considers the impact of image coordinates measurement accuracy
- Only considers error propagation due to image scale
- Exterior Orientation Parameters (EOP) and Interior Orientation Parameters (IOP) are errorless
- The left and right images are captured at the same flying height

APPENDIX VI.B COMPUTING RESOLUTION IN IMAGE FRAME

This appendix provides a step-by-step modeling of the nadir and the oblique pixel coverage on the ground for the imaging geometry illustrated in Figure VI.AppendixB.1.

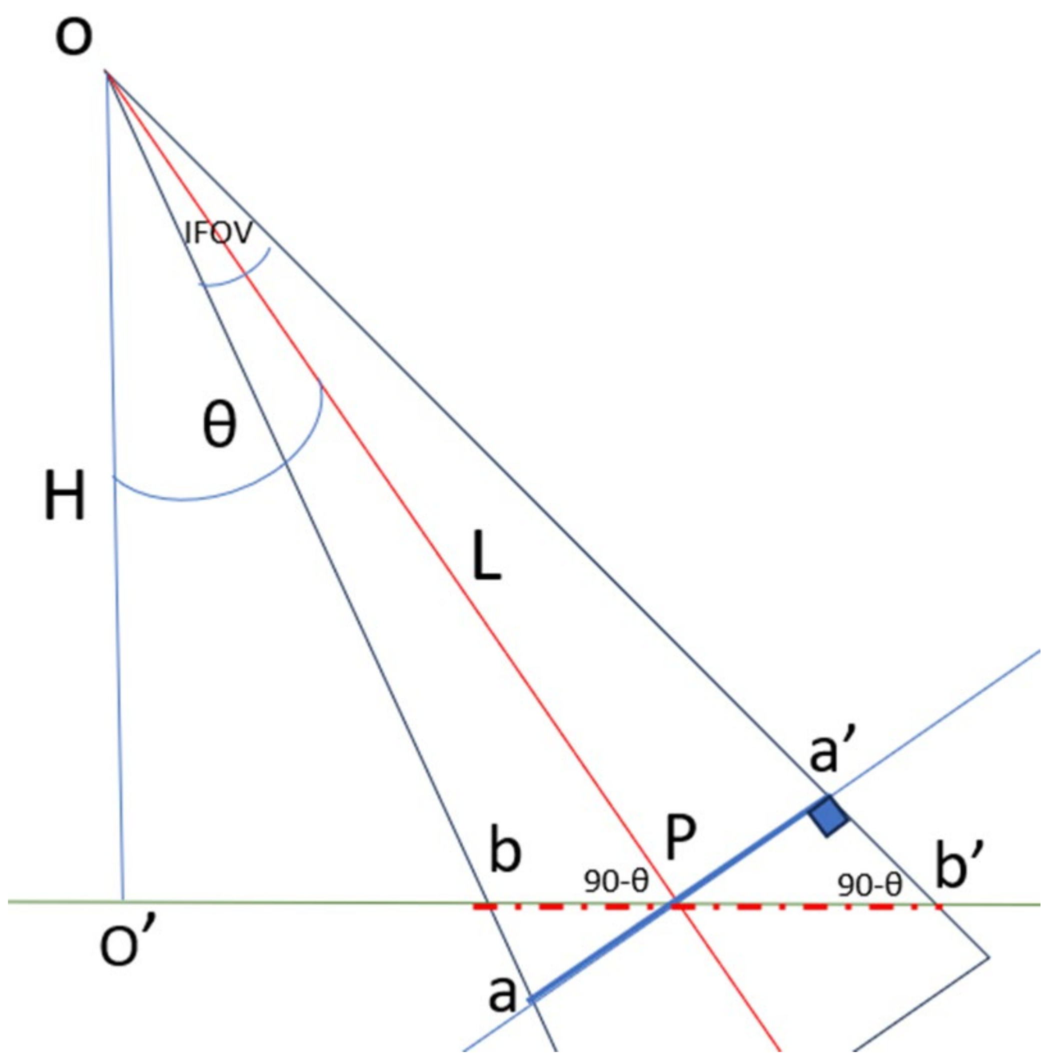

Figure VI.AppendixB.1 Geometry of Pixel in an Oblique Imaging System (courtesy: Dr. Qassim Abdullah & Michael Gruber)

VI.B.1 Computing Resolution Across Line-of-sight

Figure VI.AppendixB.1 illustrates the geometry of coverage of one pixel, where:

IFOV = Instantaneous for of view, or FOV of one-pixel
FOV = Total camera field of view
GSD_c (a-a') = GSD at object Point P cross line of sight on ground level
GSD_l (b-b') = GSD at object Point P along line of sight on ground level
H = Flying altitude above main terrain
L = Distance from camera to ground point P

θ = Pixel look angle

At near-oblique, θ = β - FOV/2

At mid-oblique, θ = β

At far-oblique, θ = β + FOV/2

β = Camera mounting angle from nadir

f = Lens focal length

CCD_{size} = CCD or CMOS physical dimension

The following formulas can be used to compute the GSD of imagery collected during an oblique imaging mission across the line of sight or a-a' at different positions in the oblique image regions: near-oblique, mid-oblique, and far-oblique:

IFOV = $\mathrm{CCD}_{\mathrm{size}} / \mathrm{f}$

L = H / cos (θ)

GSD_c at distance L = IFOV x L, or

GSD_c = IFOV x H / cos (θ), or

$GSD_c = \frac{\mathrm{CCD_{size}}}{\text{focal length}} \times \frac{\text{Flying altitude}}{\cos(\text{look angle})}$

Position O• of Figure VI.AppendixB.1: $\mathrm{GSD_{c\ nadir}} = \frac{\mathrm{CCD_{size}}}{\mathrm{f}} \times \mathrm{H}$

Position A• of Figure VI.AppendixB.1: $\mathrm{GSD_{c\ near\text{-}oblique}} = \frac{\mathrm{CCD_{size}}}{\mathrm{f}} \times \frac{H}{\cos\left(\beta - \frac{FOV}{2}\right)}$

Position O•• of Figure VI.AppendixB.1: $\mathrm{GSD_{c\ mid\text{-}oblique}} = \frac{\mathrm{CCD_{size}}}{\mathrm{f}} \times \frac{H}{\cos(\beta)}$

Position B• of Figure VI.AppendixB.1: $\mathrm{GSD_{c\ far\text{-}oblique}} = \frac{\mathrm{CCD_{size}}}{\mathrm{f}} \times \frac{H}{\cos\left(\beta + \frac{FOV}{2}\right)}$

VI.B.2 Computing Resolution Along Line-of-sight

To calculate the GSD along line of site (b-b') of Figure VI.AppendixB.1, the following assumptions were made:

1. IFOV is too small, so angles at a and a' are considered right angles, i.e., 90 degrees each
2. Same justification makes the angle at b' is equal to the angle at P or 90-θ

Therefore,

Sine (90- θ) = ½(a-a')/ ½(b-b')

½(b-b') = ½(a-a')/ Sine (90- θ)

Or,

GSD along line of sight (b-b') = (aa')/ Sine (90- θ)

Therefore,

Position A• of Figure VI.AppendixB.1: $\mathrm{GSD_{l\ near\text{-}oblique}} = \left(\frac{\mathrm{CCD_{size}}}{\text{focal length}} \times \frac{\text{Flying altitude}}{\cos\left(\beta - \frac{FOV}{2}\right)}\right) / \sin\left(90 - \left(\beta - \frac{FOV}{2}\right)\right)$

Position O•• of Figure VI.AppendixB.1 : $\mathrm{GSD}_{\mathrm{l\ mid\text{-}oblique}} = (\frac{\mathrm{CCD_{size}}}{\text{focal length}} \times \frac{\text{Flying altitude}}{\cos(\beta)}) \,/\, \sin(90-\beta)$

Position B• of Figure VI.AppendixB.1: $\mathrm{GSD}_{\mathrm{l\ far\text{-}oblique}} = (\frac{\mathrm{CCD_{size}}}{\text{focal length}} \times \frac{\text{Flying altitude}}{\cos\left(\beta+\frac{FOV}{2}\right)}) \,/\, \sin\left(90-(\beta+\frac{FOV}{2})\right)$

The oblique GSD along the line of sight can also be derived using the following equation:

$GSD\ along\ line\ of\ sight\ b-b' = \mathrm{O'b'} - \mathrm{O'b} = Flying\ Altitude\ (H)\ [tan\,(\theta + \frac{IFOV}{2}) - tan\,(\theta - \frac{IFOV}{2})]$

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