Use of GNSS in land surveying and mapping

RICS professional standard

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Lead author(s)
Dr. Christopher Pearson (School of Engineering, Newcastle University)
Prof. Stuart Edwards FRICS (School of Engineering, Newcastle University)
Dr. Nigel Penna (School of Engineering, Newcastle University)

Peer Review Group
Paul Cruddace FRICS (Ordnance Survey GB)
Barry Gleeson FRICS (WSP)
Mark Greaves MRICS (Ordnance Survey GB)
Richard Groom FRICS (Consultant)
Dr. William Kelly (Glasgow University)
Marcin Krzetowski (Network Rail)
Mark Lawton (Skanska)
Dr. Audrey Martin FRICS (Technical University Dublin (TUD))
Duncan Moss FRICS (Ordnance Survey GB)
Dr. Eugen Niculae (Technical University Dublin (TUD))
Ian Thatcher (HS2)
Dr. James Turner (HS2)

RICS standards lead
James Kavanagh MRICS (Head of Land and Resources Group RICS)

Standards delivery project manager
Helvi Cranfield

Editor
Jo FitzLeverton

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RICS standards framework

RICS' standards setting is governed and overseen by the Standards and Regulation Board (SRB). The SRB's aims are to operate in the public interest, and to develop the technical and ethical competence of the profession and its ability to deliver ethical practice to high standards globally.

The RICS Rules of Conduct set high-level professional requirements for the global chartered surveying profession. These are supported by more detailed standards and information relating to professional conduct and technical competency.

The SRB focuses on the conduct and competence of RICS members, to set standards that are proportionate, in the public interest and based on risk. Its approach is to foster a supportive atmosphere that encourages a strong, diverse, inclusive, effective and sustainable surveying profession.

As well as developing its own standards, RICS works collaboratively with other bodies at a national and international level to develop documents relevant to professional practice, such as cross-sector guidance, codes and standards. The application of these collaborative documents by RICS members will be defined either within the document itself or in associated RICS-published documents.

Document definitions

Document type	Definition
RICS professional standards	Set requirements or expectations for RICS members and regulated firms about how they provide services or the outcomes of their actions. RICS professional standards are principles-based and focused on outcomes and good practice. Any requirements included set a baseline expectation for competent delivery or ethical behaviour. They include practices and behaviours intended to protect clients and other stakeholders, as well as ensuring their reasonable expectations of ethics, integrity, technical competence and diligence are met. Members must comply with an RICS professional standard. They may include: • mandatory requirements, which use the word 'must' and must be complied with, and/or • recommended best practice, which uses the word 'should'. It is recognised that there may be acceptable alternatives to best practice that achieve the same or a better outcome. In regulatory or disciplinary proceedings, RICS will take into account relevant professional standards when deciding whether an RICS member or regulated firm acted appropriately and with reasonable competence. It is also likely that during any legal proceedings a judge, adjudicator or equivalent will take RICS professional standards into account.
RICS practice information	Information to support the practice, knowledge and performance of RICS members and regulated firms, and the demand for professional services. Practice information includes definitions, processes, toolkits, checklists, insights, research and technical information or advice. It also includes documents that aim to provide common benchmarks or approaches across a sector to help build efficient and consistent practice. This information is not mandatory and does not set requirements for RICS members or make explicit recommendations.

Glossary

Almanac	A set of parameters transmitted by each GNSS satellite that enables a receiver to predict the approximate location of the satellite. The data includes orbit information on all the satellites, clock correction and atmospheric delay parameters. This data is used to facilitate rapid satellite vehicle acquisition. The orbit information is a subset of the ephemeris data, with reduced accuracy.
Ambiguity	The unknown integer number of carrier phase cycles in an unbroken set of GNSS measurements. In GNSS processing mathematical calculations are made to compute this number, thus 'resolving the ambiguity'.
Baseline	The three-dimensional vector distance between a pair of stations for which simultaneous GNSS data has been collected and processed with static differential techniques. This is the most accurate GNSS result.
Coarse/Acquisition (C/A) Code	Two pseudo random noise (PRN) codes are transmitted by each GPS satellite, C/A and P (Precision). C/A is the simpler, non-military code, which is modulated onto the GPS L1 signal. The code is a sequence of 1024 pseudo random binary bi-phase modulations of the GPS carrier at a chipping rate of 1.023MHz, thus having a code repetition period of one millisecond. This code was selected to provide good acquisition properties.
Carrier	An unmodulated radio wave having characteristics of frequency, amplitude, phase.
Continuously Operating Reference Station (CORS)	A network of RTK base stations that broadcast corrections, usually over an Internet connection. Accuracy is increased in a CORS network because more than one station helps ensure correct positioning and guards against a false initialisation of a single base station.
Cycle slip	The loss of lock of the satellite signal by the receiver. When lock is resumed the fractional part of the measured phase would still be the same as if tracking had been maintained. The integer number of cycles exhibits a discontinuity or 'cycle slip'.
Differential positioning	Determination of relative coordinates of two or more receivers that are simultaneously tracking the same satellites. Dynamic differential positioning is a real-time calibration technique achieved by sending corrections to the roving user from one or more monitor stations. Static differential GNSS involves determining baseline vectors between pairs of receivers.
Differential processing	GNSS measurements can be differenced between receivers, satellites and epochs. Although many combinations are possible, the present convention for differential processing of GNSS phase measurements is to take differences between receivers (single difference), then between satellites (double difference), then between measurement epochs (triple difference). A single difference measurement between receivers is the instantaneous difference in phase of the signal from the same satellite, measured by two receivers simultaneously. A double difference measurement is obtained by differencing the single difference for one satellite with respect to the corresponding single difference for a chosen reference satellite. A triple difference measurement is the difference between a double difference at one epoch of time and the same double difference at the previous epoch of time.

Dilution of precision (DOP)	A computed unitless scalar value which describes the geometric contribution to the uncertainty of a GNSS position solution. For any GNSS fix a DOP value is computed. It is usually either geometric DOP (GDOP), position DOP (PDOP) or horizontal DOP (HDOP). In addition, other values exist such as vertical, time and relative DOP. See definition of PDOP for further details.
Double difference method	A method to determine that set of ambiguity values which minimises the variance of the solution for a receiver pair baseline vector.
Dynamic positioning	Determination of a time series of sets of coordinates for a moving receiver, each set of coordinates being determined from a single data sample, and usually computed in real time.
Earth-centred earth-fixed (ECEF)	Cartesian coordinate system where the X direction is through the intersection of the prime meridian (Greenwich) with the equator. The axes rotate with the Earth. Z is the direction of the spin axis.
Elevation	Height above a defined level datum, e.g. mean sea level or the geoid.
Elevation mask	The lowest elevation in degrees above the horizon at which a GNSS receiver is set to track a satellite. It is usually set to 10 degrees or 15 degrees to avoid atmospheric effects and signal interference. A lower mask angle would increase ionospheric distortion and also tropospheric effects.
Ellipsoid	In geodesy, unless otherwise specified, a mathematical figure formed by revolving an ellipse about its minor axis. Used interchangeably with spheroid.
Ephemeris	The set of data that describes the position of a celestial object as a function of time. The GNSS ephemeris is used in the processing of GNSS observations. Either the broadcast ephemeris from the satellite navigation message or a precise ephemeris calculated from GNSS tracking stations can be used, depending on application.
Epoch	A point in time that is the reference for a set of coordinates. The measurement interval or data frequency, as in recording observations every 15 seconds. In this example loading data using 30-second epochs means loading every other measurement.
European Geostationary Navigation Overlay Service (EGNOS)	A European operated, satellite based real-time differential GNSS system. EGNOS transmits a signal containing information on the reliability and accuracy of the positioning signals sent out by GPS and GLONASS.
Float solution	A baseline solution that does not fix the integer ambiguity values to whole numbers. The values are left as non-integer real numbers giving the baseline a higher RMS. than a fixed baseline. In general float solutions are not acceptable as final baseline measurements.
Galileo	The European Commission's GNSS system.
Geodetic datum	A mathematical model designed to best fit part or all of the geoid. Conventional datums depended upon an ellipsoid and an initial station on the topographic surface established as the origin of the datum. Such datums were defined by the dimensions of the spheroid, by the geodetic latitude, longitude and the height of geoid above the ellipsoid at the origin, by the two components of the deflection of the vertical at the origin, and by the geodetic azimuth of a line from the origin to some other point.

	Geocentric datums are designed to give the best possible fit worldwide rather than to depend upon values determined at an initial station. Their origin is the geo-centre of the earth.
Geoid	The particular equipotential surface that most closely approximates to mean sea level in the open oceans and which may be imagined to extend through the continents. This surface is everywhere perpendicular to the force of gravity.
Geometric dilution of precision (GDOP)	The relationship between errors in user position and time and in satellite range. GDOP = PDOP + TDOP. See PDOP.
Global Navigation Satellite System (GNSS)	The generic term for satellite navigation systems, including GPS, GLONASS, Galileo and Compass.
Height – ellipsoidal	The distance above or below the ellipsoid measured along the normal to the ellipsoid at that point. Not the same as elevation above sea level. GNSS receivers output position-fix height as the height above the ITRS ellipsoid.
Initialisation	The moment when a rover GNSS receiver in a high precision real-time dynamic system (RTK) solves the integer ambiguity and gains a real-time high precision fixed baseline solution.
Ionospheric delay	The ionosphere is a non-homogeneous (both in space and time) and dispersive medium. A wave propagating through the ionosphere experiences variable delay. Phase delay depends on electron content and affects carrier signals. Group delay depends on dispersion in the ionosphere as well, and affects signal modulation. The phase and group delay are of the same magnitude but opposite sign.
Kinematic surveying	A dynamic method of GNSS surveying using carrier phase observations in which one receiver is moving and one receiver is stationary. It is a highly productive survey method, useful for ground control or camera positioning, but is sensitive to high DOP values, multipath interference and loss of signal lock. Operational constraints include starting from or determining a known baseline, and tracking a minimum of four satellites. One receiver is statically located at a control point, while others are moved between points to be measured.
Multipath errors	Signals can arrive at a GNSS receiver either by direct line of sight or can be reflected off nearby objects (hills, buildings, etc.), in which case the differences in path length will cause interference at the antenna and corrupt the pseudorange measurements and subsequent positional reliability. (An interference similar to ghosting on a television screen).
Narrow lane	A baseline solution that is a linear combination of the L1 and L2 frequencies. It is often an intermediate solution used for statistical testing in the process of obtaining a final L1 or iono free fixed solution.
Network RTK	The networking of GNSS base stations to enable real-time corrections to be generated and transmitted to users anywhere in the area covered by the base station network. Users receive GNSS corrections from a central source, not directly from individual base stations. The systems work by using the GPS observations at known network stations to model the unknown bias sources across the network area. From these, positional corrections can be

	generated and delivered to the rover as either a set of multiple reference stations or as the corrections that would be generated from a 'virtual' base station adjacent to the user. Actual raw base station GPS data or virtual GPS observation data is also transmitted to the rover.
Point positioning	A position produced from one receiver in a stand-alone mode.
Position dilution of precision (PDOP)	PDOP is a unitless scalar value expressing the relationship between the error in user position and the error in satellite position. Geometrically, for four satellites PDOP is proportional to the inverse of the volume of the pyramid formed by unit vectors from the receiver to the four satellites observed. Values considered good for position are small, say 3. Values greater than 7 are considered very poor. Thus, small PDOP is associated with widely separated satellites. PDOP is related to horizontal and vertical DOP by $PDOP^2 = HDOP^2 + VDOP^2$. Small PDOP is important in dynamic surveys, which are sensitive to larger PDOP values, but much less so in static techniques.
Pseudo random noise (PRN)	PRN is a sequence of binary digits that appear to be randomly distributed. This is used in the GNSS C/A and P codes, with each GNSS satellite transmitting a unique PRN. GNSS receivers use this PRN to identify which satellites they are tracking. The important property of PRN codes is that they have a low auto correlation value for all delays or lags except when they are exactly coincident. Each NAVSTAR satellite has its own unique C/A and P pseudo random noise codes.
Pseudorange	The apparent distance from a satellite to the phase centre of a GNSS receiver antenna. This is computed from the C/A or P code which gives a signal propagation time. This time can then be multiplied by the speed of light to give an apparent distance, which is not the true distance. Pseudorange differs from the actual range by the amount that the satellite and user clocks are offset, by propagation delays, and other errors. The apparent propagation time is determined from the time shift required to align (correlate) a replica of the GNSS code generated in the receiver with the received GNSS code. The time shift is the difference between the time of signal reception (measured in the receiver time frame) and the time of emission (measured in the satellite time frame).
Radio Technical Commission for Maritime Services – State Space Representation (RTCM SSR)	A high accuracy data output from International GNSS Service (IGS) that allows users to access information on satellite orbit errors, satellite clock errors, satellite signal biases, ionospheric propagation delays and advances, and tropospheric delays.
Receiver Independent Exchange format (RINEX)	A set of standard definitions and formats to promote the free exchange of GNSS data and facilitate the use of data from any GNSS receiver with any software package. The format includes definitions for three fundamental GNSS observables: time, phase, and range.
Reference frame	The realisation of any particular coordinate reference system by the measurement of points using survey instruments. There can be several realisations of any system as survey techniques and methods change.

Reference system	A mathematical definition of the particular coordinate system, including the origin, scale position and orientation of the reference ellipsoid.
Relative positioning	The process of determining the relative difference in position between two points with greater precision than that to which the position of a single point can be determined. Here, a receiver (antenna) is placed over each point and measurements are made by observing the same satellites at the same time. This technique allows cancellation (during computations) of all errors which are common to both observation sets, such as satellite clock errors, satellite ephemeris errors and the majority of propagation delays, etc.
Root mean square (error) (RMS, RMSE)	In general, when accuracies or tolerances have been specified, they refer to vector errors and are defined statistically as root mean square errors (RMSE), or as maximum tolerances. The RMSE is equivalent to a 67% tolerance, and a 90% tolerance is 1.65 times the RMSE when a representative sample of points is tested. Thus an RMSE of ± 0.01m indicates that in a representative sample of 100 points, it is expected that not less than 67 will be correct to better than ± 0.01m, and not less than 90 points will be correct to better than ± 0.016m. Any errors exceeding three times the RMSE, in this case ± 0.03m, can be regarded as mistakes.
Sigma (one sigma)	The 68th percentile or one standard deviation measure in a statistical population.
Static positioning	Positioning applications in which the positions of static or near static points are determined.
Tropospheric correction	The correction applied to the measurement to account for tropospheric delay. This value is obtained from a model such as that of Hopfield.
WGS 84 World Geodetic System (1984)	The geocentric datum used by GNSS since January 1987. It has its own reference ellipsoid. WGS 84 is fully defined in publications by the US. National Imagery and Mapping Agency (NIMA).
Wide lane	A linear combination of L1 and L2 observations (L1–L2) used to partially remove ionospheric errors. This combination yields a solution in about one-third the time of a complete ionosphere-free solution.

1 Introduction

This standard forms part of a series of specifications and guidelines intended to assist those connected with the requesting, purchase and production of surveys and mapping material at all scales, by spreading good practice and seeking to avoid duplication of effort.

This document has been written primarily to provide:

1. the surveyor with a set of practical operational guidelines, which can be used when undertaking any survey that includes GNSS techniques. Sufficient information is also included to allow the surveyor to generate a set of GNSS survey procedures applicable to a survey task required by the client
2. the client, or purchaser of spatial information generated from a GNSS survey, with sufficient information to write a task-specific specification for a GNSS survey, which sets out the accuracy requirements, products and a scope of work, from which the surveyor can accurately produce a bid for the survey.

Unlike survey specifications, this document is intended to provide best practice guidance only and should not be incorporated verbatim into the text of individual contracts. However, the wording of individual paragraphs and the surveyor and client checklists can be so used, and copyright provisions are waived solely for this purpose.

There are other RICS publications related to the full range of land surveying services such as:

- Measured surveys of land, buildings and utilities
- Earth observation and aerial surveys

These survey guidelines are mainly concerned with the land survey applications of GNSS. Many of the methodologies and technology are the same for hydrographic or aerial positioning, however these specific areas are covered in more detail within other documents listed below as well as additional complementary guidance documents.

Additionally, there are further publications relating to GNSS positioning or related fields that complement this guidance and provide more detailed information on particular topics, such as:

- GNSS Network RTK Surveying in Great Britain (2015) The Survey Association
- Commercial Network RTK GNSS services in Great Britain (2012) The Survey Association
- Railway Surveying (2017) The Survey Association
- Small Unmanned Aircraft Surveys (2016) The Survey Association
- Offshore Hydrographic Survey (2017) The Survey Association
- Topographic, Engineering, Land and Measured Building Surveying – Strategy and General (2019) Network Rail
- Drones: Applications and compliance for surveyors (2019) RICS
- A guide to coordinate systems in Great Britain (2020) Ordnance Survey

With the rapidly advancing technological edge of GNSS surveying, not all aspects of current research and development can be covered by these guidelines. Most GNSS surveying products commercially available at the time of publication are covered in this edition.

2 The role of GNSS in surveying

GNSS is now a widely used three-dimensional measurement system that uses radio signals emitted from satellites to determine position. There are four primary constellations of satellites that contribute to the GNSS network:

- United States' Global Positioning System (GPS)
- Russian Global Navigation Satellite System (GLONASS)
- European system (Galileo)

- Chinese system (BeiDou).

Further to these global constellations, there are two more regional systems operating in India (IRNSS) and Japan (QZSS) that provide supplementary observations when in those regions. The majority of modern GNSS receivers are capable of acquiring observations from multiple different satellite systems. The use of additional satellite constellations will likely enhance any positioning technique due to the added redundancy in the solution. However, the level of enhancement will vary depending on a number of factors and these are discussed further in section 3.3.

Each satellite constellation transmits signals on two or more frequencies in the L band of the microwave spectrum (1000–2000 GHz). Each signal contains ranging codes and navigation data to allow receivers to determine the travel time from the satellite and thus ranges to satellites, and ultimately receiver position. The frequency of the signals are unique to each constellation and can be seen in Figure 1.

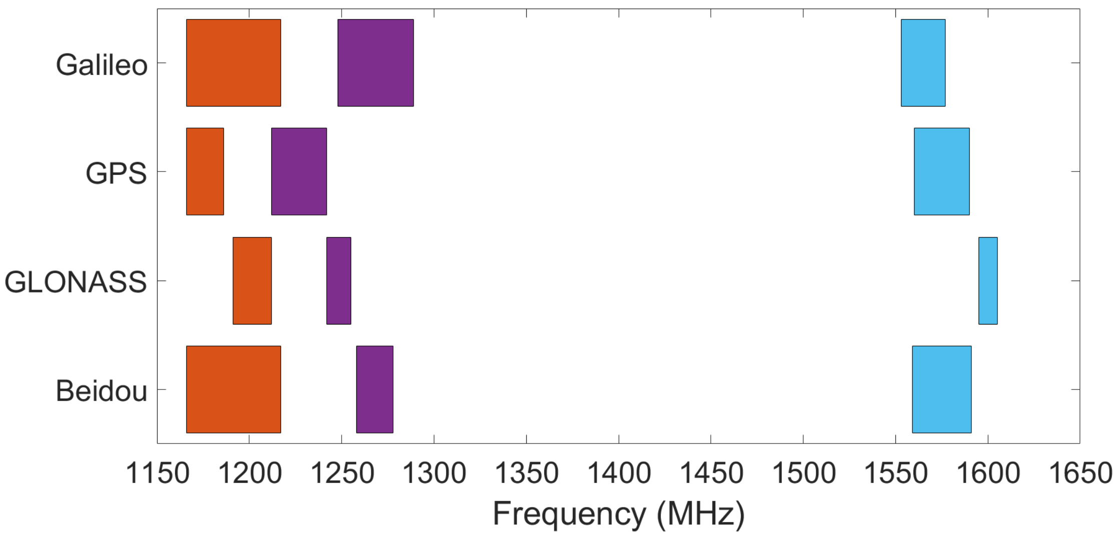

Figure 1: GNSS Frequencies (Red – E6 (Galileo), L5 (GPS), G3 (GLONASS), B3 (Beidou), Purple – E5 (Galileo), L2 (GPS), G2 (GLONASS), B2 (Beidou) , Blue – E1 (Galileo), L1 (GPS), G1 (GLONASS), B1 (Beidou))

All four global satellite constellations have unique Earth-Centred Earth-Fixed (ECEF) coordinate reference frames. GPS utilises the World Geodetic System 1984 (WGS84), GLONASS operates in Parametry Zemli 1990 (PZ-90), Galileo employs the Galileo Terrestrial Reference Frame (GTRF) and BeiDou uses the BeiDou Coordinate System (BDC). More information about the importance of different coordinate systems and the transformations between them can be found in section 5.

2.1 GNSS in perspective

GNSS has been used extensively by land surveyors since the late 1980s, primarily for geodetic control networks and for photogrammetric control. As systems have become more compact, more technologically advanced, easier to use and with a full complement of satellites across multiple constellations, the diversity of surveying applications has increased substantially. GNSS systems are now available for many surveying and mapping tasks, including establishing control, setting out, real-time deformation monitoring, on-board sensor positioning for aerial surveys; the list is continually growing.

GNSS is a tool for fixing positions and a variety of approaches are available, depending on the required accuracy of the points to be fixed. They range from low-cost systems with a positional accuracy of a few metres, to high-cost geodetic survey systems able to determine positions to the sub-cm level. For survey accuracies, it is essential that hardware and software specifically designed for survey applications are used; these should always be utilised in the manner for which they were designed.

WGS84 serves as a global coordinate reference frame for GPS with similar reference frames for each GNSS constellation. However, these reference frames are imprecisely realised especially through the broadcast satellite ephemeris. For survey applications a more accurate and up to date version of the constellations' reference frames exists. Internationally this is termed the International Terrestrial Reference System (ITRS) and regional derivatives also exist – ETRS89 for Europe, for example.

The ITRS subsequently has multiple realisations, International Terrestrial Reference Frames (ITRFyyyy) where yyyy relates to the year (epoch), which use the latest data and techniques to realise the ITRS in as precise and accurate a manner as possible. Consequently, each realisation can differ from previous versions by up to a few cm.

At point of publication the latest realisation of the ITRS is the ITRF2014 from 2014, but this will soon be superseded by ITRF2020 (since 2000 ITRF realisations have used 4-digit year numbers). For clarity, ITRS is used throughout this document, but the current epoch version, applicable to location, should be substituted whenever it is read. Note that the coordinate grids and datums used for survey and mapping projects can differ, and it is usual to transform GNSS coordinates to a local or national system. It should also be noted that WGS84 is not fixed to any particular land mass and that the process of global plate tectonics can lead to co-ordinate movement over time. An extensive global collection of transformation parameters and reference frameworks can be found in the EPSG Geodetic Parameter Dataset.

A number of public domain information and GNSS data sources are available specifically for GNSS land surveying usage, in particular the operational state of the system and information from various national mapping agencies. GNSS data is also available in the form of base station data and precise orbital elements. Many national mapping agencies and commercial organisations have developed active and passive reference stations and network RTK reference station networks.

One of the basic tenets of surveying is 'working from the whole to the part', and GNSS is no different from any other form of survey in this respect. Points of detail are measured within a control framework just as in traditional surveys. GNSS is not a total, unqualified, revolution in land surveying: optical instrument techniques still have an important role. There is a wide variety of survey applications which are unsuitable for GNSS, mainly due to the limiting factor that GNSS requires a clear view of the sky to receive satellite signals and that GNSS techniques do have upper limitations that preclude use for tasks requiring the highest engineering accuracy and precision.

2.2 Survey types

There are essentially three types of survey when using GNSS techniques. The three types can be split conveniently into different accuracy bands:

- control surveys – high accuracy
- detail surveys – medium accuracy
- low precision positioning – low accuracy.

2.2.1 Control surveys

A GNSS control survey is used to form the main coordinate framework for a project, as in a classical survey. Control surveys are typically at sub-cm accuracy and usually conducted using one of the static techniques outlined in section 3.1. The number of stations, their location and spacing will be determined by the purpose of the control, the required accuracy of the eventual survey and the type of GNSS equipment available.

The highest quality control surveys will require a geometrically well-balanced design scheme, which should include several redundant observations.

The requirements to achieve the highest quality might include:

- three independent sessions, being observed over each baseline at different times to different satellite constellations. This reduces the likelihood of external influences creating systematic errors such as poor satellite geometry and provides redundancy to the observations
- four or more control points, either newly established or from local continually operating stations provided by a national or international mapping service
- stable survey markers
- control stations located in areas with good sky views with no obstructions above 15 degrees of elevation
- terrestrial (traverse and level) observations as assurance checks between control markers (if possible)
- choke ring antennas to minimise multipath effects (if possible)
- antennas oriented to true north at all stations to eliminate eccentricity effects and to enable the accurate application of azimuthal-dependent phase centre models
- antenna phase centre models, e.g. use of National Geodetic Survey (NGS) absolute multi-constellation phase centre models

- where possible, sessions planned to enable a minimum of five satellites to be observed, avoiding high peaks in position dilution of precision (PDOP). Multi-constellation GNSS can be used to ensure additional redundancy in the solution
- log all receiver and antenna types used in the scheme, to allow for antenna phase centre variation calculations to be made
- appropriate session duration and epoch separation as outlined in Table 1 (section 3.1.4)
- use precise orbit and clock products where appropriate to remove errors in the broadcast ephemeris (section 4.2)
- ensure all equipment is fully calibrated
- detailed site record logs including witness diagrams, antenna heights, height measurement points, filename, point ID, operator names, epochs and epoch rates, environmental and meteorological observations.
- use of postprocessing software.

2.2.2 Detail surveys

GNSS provides an excellent tool for quickly, accurately and reliably positioning points of detail, for example the points or features that may need to be mapped as part of the survey, within the confines of an area surrounded by the control survey. GNSS techniques do have upper limitations that preclude use for tasks requiring the highest engineering accuracy and precision. GNSS detail surveys typically have a requirement for accuracy of 1-10 cm. However, some applications require lower accuracies in the 10 to 30 cm range. Therefore, it is important that the surveyor decides early on which type of GNSS data capture technique is most applicable to their locality and/or survey specification – national RTK network, single baseline or own base station, for example.

There may be many instances of smaller areas of detail in a mapping project where traditional survey methods are more appropriate. Such methods can be quicker and more accurate, especially in urban areas where sky view is limited and multipath levels are high.

For the majority of applications of GNSS detail surveys, dynamic positioning methods such as those outlined in section 3.2 may be most appropriate. The exact method used will depend on the accuracy of the specification in the project. For higher accuracy applications requiring horizontal accuracies of 1-10cm, the following field procedures should be used as best practice guidelines.

Installation

- Collect detail points in the control framework created for the project.
- Dynamic relative techniques should be limited to 10–15 km baselines.
- If further base station set-ups are used (for example in a long linear survey), these should be tied as part of a main network to other base stations used. The survey should also include common points, observed from each adjacent base.
- Take antenna phase centre variations into account if different types of antenna are used.
- If any new tertiary level control is observed, each new station should be

coordinated twice, once from each of two base control stations, to ensure an independent check.
- Both receivers should maintain lock on at least four satellites throughout the session. This technique should only be used in areas where loss of lock is likely to be minimal, e.g. open spaces, clear of vegetation and structures.
- Use alternative traditional surveying techniques in severe areas of sky view obstructions.
- Initialise the GNSS receiver in an area clear of obstructions and note any new initialisation due to loss of lock on the satellites.

Observation

- When starting the survey, once the receiver has completed initialisation it is good practice to make a separate check on a known station, to ensure the receiver has the correct integer solution and the base station has the correct coordinates.
- When a loss of lock occurs during the survey, a previously coordinated point should be revisited in order to check the re-initialisation of the receiver.
- Monitor or set masks for coordinate accuracy and Geometric Dilution of Precision (GDOP) to ensure values are maintaining the precision required for the specification.

Data processing

- Log all raw data in real-time surveys to allow re-computation in post processing if any issues arise. Ensure that data is archived appropriately.

Lower order detail surveys where accuracies can be up to 1m may use differential GNSS as outlined in section 3.2.2. These can be real time or post-processed, and in all cases the surveys are not subject to the same loss of lock criteria as higher-order surveys. The guidelines below could be followed for fieldwork.

- Occupy known points during the survey – these could be either national control points, or other control points in the survey area measured using a higher-precision method.
- Observe map check points if data is to be fitted to a background map. This will ensure the data is consistent with a map background and will allow the accuracy of the background map to be assessed.
- Ensure the positional data logged includes the number of satellites, DOP values, estimated precision and the standard deviation of any multiple differential GNSS (DGNSS) positions logged for a point.
- Try to use the equipment where clear sky views are available. Despite manufacturers’ claims that DGNSS equipment will work in wooded and other enclosed areas, it is best practice to use other traditional survey methods in such areas to provide offset points in low-precision applications.

2.2.3 Low precision positioning guidelines

Lower precision GNSS positioning frequently uses a single receiver, possibly receiving real-time DGNSS corrections or logging data for later post-processing, and is different to control and detail surveying. As a result, the accuracy for positioning is generally at the level of one to a few metres, rather than at a few cm. This type of survey would normally be undertaken for navigation or for surveying features at the metre level to input into a CAD package or geographic information system (GIS).

Due to lower accuracy requirements, the quality control of the field procedures for positioning tasks is far less rigorous than for control or detail surveys. The main issues to be aware of while undertaking the survey are:

- ensure the file name for any logged data is noted and booked in the field with the point names
- use GNSS instrument waypoint storage to log points in the field
- keep a copy of all data in ITRS for later transformation if required
- real-time systems for data capture allow operators to ensure sufficient data has been collected
- when capturing features to be presented on a map background ensure to survey some well-defined features such as road junctions to provide check points to assess map accuracy. If these features do not fit the background map, do not fit the data to the map as in some cases it may be the map that is in error
- Satellite-Based Augmentation Systems (SBAS) such as the European Geostationary Navigation Overlay Service (EGNOS) can be used to improve positioning accuracies by correcting for a large part of the ionospheric error in the observations when using single frequency receivers.

3 Survey methods

GNSS survey techniques can be separated into the following two key methods: static surveys and kinematic surveys. Within both methods each technique has a varying level of likely precision, optimum occupation time and potential baseline length required. The suggested precisions given in the following sections rely on optimal conditions such as good satellite availability, good GDOP, normal ionosphere and tropospheric conditions, and low levels of multipath. However, in sub-optimal conditions these likely precisions will degrade and alternatives such as longer occupations times would be required in order to maintain precision. The suggested precisions given in the tables are also based on using observations from GPS satellites only rather than a multi constellation solution. The potential impact of multi constellation positioning is discussed in section 3.3.

The likely precision of each GNSS survey method is stated. However, precision is not the same as accuracy. Precision is the ability to repeat a measurement and get the same answer. Accuracy is a measure of how close the measurement is to the true dimension. The precisions given in Table 1 (section 3.1.4) and Table 2 (section 3.2.3) are of the baseline components in the GNSS coordinate system at one sigma (σ). (This is rarely the final coordinate system for the survey.) One of the significant potential sources of error in a GNSS survey is the transformation process. However, if these best practice guidelines are followed, such errors will be minimised. If this is the case, the values stated for precision will reflect those achievable for final coordinate system accuracy.

In many of the following positioning methods data from a Continually Operating Reference Station (CORS) network is used. Hence, the absolute accuracy of the user position is highly dependent on the accuracy of the stations in the CORS network. This may vary between networks but is typically 1-2 cm depending on how much data has been used to compute the coordinates of the CORS network. The accuracy of the CORS network being utilised is not considered in the likely precisions of each GNSS method stated. Therefore, the likely precisions quoted are relative to the local CORS network, which should be considered when assessing overall absolute accuracy.

3.1 Static positioning

In a static GNSS survey, the antennas and receivers remain fixed over control points during the period of observation. Where GNSS observations are to be used solely to fix plan position, the site chosen for the control station need not be optimum; however, when the accuracy of the height component is required to be better than 40 mm, site selection is more important. In this case the reliability of the coordinates will be adversely affected by trees blocking the signals and by multipath from buildings and reflective objects. It is best practice to select sites for GNSS control observations that have a completely clear sky view above 10° elevation and no objects such as buildings that might cause multipath (alternatively, Virtual RINEX data can be used, which replaces the need to set up a base station if the area is within a CORS network). Geoid corrections should be applied to convert GNSS ellipsoidal heights to orthometric heights and, for the highest millimetric accuracies, it is also advisable to connect the survey stations using closed loop levelling.

In this section a number of static survey techniques are explored, in descending levels of precision. Table 1 in section 3.1.4 summarises the levels of precision expected in each method.

3.1.1 Static baseline

Dual-frequency static (for baselines of less than 100 km) methods are most suitable for control surveys and afford the highest precision (sub-cm) achievable with GNSS. It requires the simultaneous observation at two or more stations of GNSS data from four or more common satellites. The length of occupation required varies depending on the baseline length as outlined in Table 1 (section 3.1.4). Carrier phase and code measurements are made on all available frequencies. A baseline vector is computed after the observations have been logged, using a differencing technique.

The baseline computation is a series of processing steps. Triple differencing is often used firstly to determine and correct cycle slips in the data. The baseline estimation then proceeds, using a double difference of the phase observables. In modern processing methods, highly advanced statistical testing is carried out to determine the best candidate in a search area, for integer ambiguity determination. A series of solutions can be determined using combinations of the phase observables such as 'wide lane' or 'narrow lane'. These are often used as a step to confirming the final solution, which is either 'iono free fixed' or 'L1 fixed', according to baseline length.

A successful outcome from each of these processing steps is most likely when:

- there is a large amount of good quality data
- the antennas remain stationary during the observations
- there is a significant geometry change in the satellite constellation.

If these conditions all exist in the high-precision static method, the best coordinates are achieved. The high volume of data is used to not only solve for the integer ambiguity as mentioned above, but also to solve for other unknowns in the mathematical process of computing the baseline.

In addition, other errors such as the delays caused by the atmospheric refraction can be solved by the software using empirical models or parameter estimation. In essence, high-precision surveys require the removal of as many systematic errors as possible. This includes the use of the precise ephemeris, as it reduces the metre-level orbital errors that are present in the broadcast ephemeris. The actual ephemeris is available in several forms (rapid, ultra-rapid, precise) as described in section 4.2.

It is important to ensure that the make of the processing software and the version being used is capable of importing a precise ephemeris and using it correctly. When computing baselines of longer than about 80 km, 'scientific' software (e.g. Astronomical Institute of the University of Bern – Bernese GNSS Software) that incorporates algorithms to compute ocean tide loading and tropospheric delay corrections, should be used.

Epoch settings or 'epoch intervals' are the rate at which data is logged in the memory of the receiver. In the differencing process, to compute the baseline solution the observations from one epoch are differenced with those from the next. The interval appropriate for any particular survey depends on a variety of factors. These include the type of baseline processing software, the amount of memory available in the receiver, and the density of point data needed for the survey. In a static survey, as the antennas remain fixed, there is no need for an epoch setting of less than 15 seconds. Whatever epoch setting is selected, it is imperative that all receivers used in the survey are set to the same interval.

3.1.2 Rapid static

For applications requiring precision of approximately 1 cm in plan and 1-2 cm in height the rapid static approach can be used. The rapid static survey method is similar to high precision static surveys, but occupation times are reduced, depending on conditions. The key technical differences between high-precision static and rapid-static are:

- the processing software should have sophisticated processing algorithms to allow computation of the baselines
- it is of greater importance that the survey data should be virtually free of cycle slips, multipath and interference
- good satellite geometry is critical
- baselines are limited to a maximum of 40 km.

These more onerous conditions are all required, even though rapid static gives only a medium precision outcome, because a smaller amount of data is used to estimate and obtain the correct baseline solution.

The method works by using a higher epoch rate of five or ten seconds in the differencing process. This allows the same number of differences to be computed for a 20-minute observation of five second epoch, as for a one-hour observation of 15-second epoch, assuming the same number of satellites, while still allowing for some geometry change. Thus, the mathematical solution of the baseline can be found. However, the presence of cycle slips, multipath and little change in satellite geometry, introduces noise to the solution. In the worst case, the limited data will fail statistical testing, and a fixed integer solution will not be computed by the software.

In most cases this will result in the necessary observation of data to compute the baseline. The use of float solution baselines and advanced processing techniques to accept marginal solutions is not recommended. In general, the limited data allows baseline computation to an acceptable L1 or iono-free fixed solution. However, assumptions are made by the mathematical models used in the processing software and this leaves some system errors in the solution.

Hence, the solutions are not as precise as in the high-precision static case.

3.1.3 Static network RTK

With network RTK, instead of using a base station set up over a known point collecting simultaneous observations, the user subscribes to a network RTK correction service that provides the base station data. Tests in Great Britain have shown that it is possible to obtain position to RMS 10–15 mm and height to RMS 15–30 mm from network RTK observations. Where these are used to fix survey control stations, as a minimum observations should be made using two sessions of three minutes separated by at least 20 minutes, preferably under different satellite configurations.

3.1.4 Precise Point Positioning (PPP)

PPP is the optimal approach for standalone positioning (where no CORS network is available). PPP traditionally requires a single dual frequency, code and carrier-phase GNSS stationary receiver recording static observations for several hours or more to attain cm-level precision and accuracy. The data can be processed in scientific software along with precise satellite orbits and clocks as described in section 4.2. While it is possible to conduct PPP positioning using real-time or rapid orbits and clock products, the best accuracies and those quoted in Table 1 below are derived using the final orbits available within 12-18 days of data collection. For shorter observation periods of less than 5 hours it is recommended that observations from at least two GNSS constellations are used to maintain precision levels.

PPP processing software is available from multiple different sources depending on the level of control and detail you require. Highly scientific proprietary software such as JPL's (Jet Propulsion Laboratory) GipsyX and AIUB's (Astronomical Institute of the University of Bern) Bernese allow users to control numerous processing parameters and assess numerous parameters in a detailed manner with in-depth analysis of the final coordinates.

Processing using this software offers the potential for the most precise and reliable coordinates, however it requires expertise in GNSS processing and is more time consuming. Free online alternatives are available such as MagicGNSS, CSRS-PPP and GAPS where RINEX files can be upload for automated processing. These online services offer fast and free positioning options but users should be aware of potential error sources and the potential impact on the reliability of any solution obtained from these services.

Technique	Observations	Baseline length	Occupation time	Accuracy
Static baseline	Dual frequency	20 km	>1 h	H 5 – 10 mm V 10 mm
		30 km	>2 h	
		50 km	>4 h	
		100 km	>6 h	
Rapid static	Dual frequency	<10 km	>5 m (5 s or 10s epoch rate)	H 10 - 15 mm V 10 – 20 mm
		<15 km	>15 m (5 s or 10s epoch rate)	H 10 - 20 mm V 20 – 30 mm
Static network RTK	Dual frequency	< 40km	2x >3 min separated by >20 min	H 10 - 15 mm V 15 – 30 mm
PPP	Dual frequency	N/A	24 h	H 10 mm V 10 – 15 mm
			6 h	H 15 mm V 15 – 25 mm
			1 h	H 50 mm V 60 – 100 mm

Table 1: Static positioning – potential accuracies in horizontal (H) and vertical (V). Accuracies are given as one sigma (σ)

3.2 Kinematic surveys

Kinematic surveys provide the highest production rate for all the GNSS methods. While rapidly generating coordinates, the precision obtained is not as high as by static techniques. This is because in kinematic techniques, most random measurement and GNSS system errors are absorbed in the coordinates. This can be contrasted with static methods, in which they are absorbed in the residuals after a network adjustment. Kinematic surveys can be post processed or carried out in real-time, with the addition of a suitable communications link. It is critical, therefore, when using real-time solutions that the GNSS receivers have the correct firmware loaded for the chosen real-time method. The various kinematic survey approaches outlined below are in descending levels of precision and summarised in Table 2.

Technique	Observations	Baseline length	Occupation time	Accuracy
Relative kinematic	Dual frequency	1 km	5 seconds	H 10 – 20 mm V 15 - 30 mm
		15 km	1 minute	
PPP-RTK	Dual frequency	<50 km	5 minute	H 20 – 50 mm V 30 – 60 mm
		<250 km	20 minutes	
DGNSS	Dual frequency float	20 km	1 – 15 minutes	H 0.1 – 0.4 m V 0.2 – 0.8 m
	Phase smoothed code			0.4 – 1 m
	L1 code	100 km	1 minute	H 1 – 5 m V 2 – 7 m

Table 2: Kinematic positioning – Potential accuracies in horizontal (H) and vertical (V). Accuracies are given as one sigma (σ)

3.2.1 Relative kinematic

For the highest precision kinematic surveys, the methods of network RTK, on-the-fly kinematic and Post Processed Kinematic (PPK) can be used. The basic technique is the same as static network RTK: to keep one receiver fixed at a known control station (base) while one or more other receiver(s) (rovers) move around the site observing the same satellites. The survey can tolerate periodic loss of lock during the survey, as the 'integer ambiguity' can be determined while the rover is moving to the next point. Each technique results in baseline vectors from the base station to each station visited by the rover. These techniques can be used for surveys requiring detail, or coordinates to be captured at an accuracy of a few cm. In such surveys, as the receiver at which the baseline solution is being calculated is moving, a short interval epoch setting is needed. Typically, a sampling rate of one or two seconds is used for standard detailing for land surveys. Depending on application such as photogrammetric camera positioning epoch settings below one second are typically used.

With these rapid epoch settings, differencing techniques are used with statistical search methods to quickly compute the initialisation baseline solution from the data. In the on-the-fly technique, the most advanced baseline solution methods are used to determine the baseline initialisation and then to test it, to ensure it is the correct one.

As only one to two minutes of data are used for this initialisation process, the data should be as free as possible from multipath and cycle slips, otherwise incorrect baseline solutions will result. Thus, when initialising on-the-fly surveys it is important to move to a location that has an open view of the sky. It is also good practice to reoccupy previously surveyed points after a new initialisation, as this proves that the new solution is the correct one.

Once the initial baseline is computed, the coordinates of the roving receiver are computed for each epoch. The integer ambiguity of the baseline is known from the initialisation process and thus the new baseline solution at each epoch can simply be determined from the change in the satellite observations. The changes are due to the satellite movement and the receiver movement. From the ephemeris the movement of the satellites is known, hence the movement of the receiver is solved from the mathematics in the processing software. This is why only a few seconds of data are required at each new survey point.

For applications that do not need solutions in real-time network RTK, service suppliers may provide a post-processing service for high-precision kinematic GNSS. Additionally, surveyors in some areas of the world may find free online post-processing services useful, such as RTKLIB or GAMP.

For real-time applications, high-speed communications links are used to transmit the base data to the rover. It is common for manufacturers to use UHF or VHF links integrated into base and rover complete kits. These are often supplied with the base in a rugged case with a GNSS receiver, radio, modem, power supply and antenna, all cabled together ready for use with the rover. These can either be in a rucksack or on a pole with equipment mounted on or inside it. Base data can also be transmitted to the rover by GSM or GPRS over a cellphone network.

3.2.2 Precise Point Positioning Real Time Kinematic (PPP-RTK)

PPP-RTK combines the advantages of PPP and RTK to allow centimetre level positioning with the use of Integer Ambiguity Resolution (IAR). As for float PPP, PPP-RTK requires dual frequency observations with precise satellite orbit and satellite clock products, but with the addition of satellite phase biases as well as ionospheric and tropospheric corrections to improve convergence times. These satellite phase biases and atmospheric corrections computed from a CORS network allow this single receiver approach to conduct IAR. If successful, this can reduce convergence times and improve coordinate precision when compared to standard PPP.

There are multiple different mathematical approaches that have been developed to conduct PPP- RTK which differ slightly in corrections that are transmitted to the user as outlined in *Review and principles of PPP-RTK methods* by Teunissen, P.J.G. and

Khodabandeh, A. (2015). However, all approaches require just a single user receiver with a separate large scale CORS network to compute correction values. These transmitted corrections result in double differenced observations and hence enabling IAR and centimetre level positions within as little as a few minutes or potentially less than one minute. However, convergence times depend on the accuracy of the atmospheric models, which degrade with distance. Hence, increased convergence times can be expected when moving further away from the CORS network as shown in Table 2.

Correction services can be categorised as global, regional (continental) or nationwide and corrections can be provided by satellites such as Galileo for global services or geostationary satellites for small areas. These corrections have been standardised into an Radio Technical Commission for Maritime Services – State Space Representation (RTCM SSR) format.

3.2.3 Differential GNSS (DGNSS)

For low accuracy applications such as detail surveys that only require sub metre accuracy post processed DGNSS can be used. DGNSS works similarly to kinematic network RTK but rather than requiring a dual frequency and ambiguity fixed solution it can work with L1 code, dual-frequency float or phase-smoothed code solutions. These systems vary in the achievable precision. Standard systems using corrections to the pseudoranges can be used to obtain precisions of two to five metres. However, more accurate systems using real-time smoothing of the data by observations of the carrier phase will achieve a precision of between 0.4 and 1 metre, according to operational considerations. Dual-frequency data techniques are also employed, which obtain precision down to the 0.1m-0.3m level.

For standard DGNSS the method is the same as outlined above. However, there is an international standard (RTCM SC-104) for the format of the transmitted pseudorange corrections. As well as setting up your own base station, corrections can be obtained from either free-to-air or fee-based satellite, phone or radio-based services.

In the carrier-smoothing method, several consecutive epoch observations are needed to obtain the system precision. This is because the differences in the measured pseudorange corrections and rate of correction are compared with the actual carrier phase differences between epochs. Thus, for this to be carried out precisely, several epochs of carrier data with no cycle slips are required. Typically, one-second epochs are used for GIS asset surveys. In particular the correct solution can be checked by occupation of control points or by checking map features which can be located on background maps in loggers.

Dual-frequency techniques are also used, but unlike RTK where integer ambiguities are fixed to give few-cm precision, the integers are kept to float so the end precision is at the 0.1m – 0.3m level.

3.3 Impact of multi-GNSS

The emergence of alternative GNSS systems to GPS during the 21st century has opened up opportunities for surveyors to potentially enhance their positioning solutions in terms of convergence times, overall accuracy and reliability. Many receivers on the market today are capable of receiving observations from two or more of these GNSS constellations. This increase in observations can potentially allow solutions where previously it was not possible, or enhance positions by increasing redundancy levels in any given solution.

Multi-GNSS positioning can improve the precision of the solution in any positioning technique. The level of potential improvement is greatest in sub-optimal conditions, whether that be due to the number of satellites visible, the baseline length or observation window length. In these challenging conditions, the additional observations offered by supplementary satellite systems allow precision levels to remain high despite the poor conditions.

Poor satellite availability can both reduce the levels of redundancy in any solutions and also negatively affect the GDOP, hence weakening the solution especially in the up component. For network RTK positioning in severe environments such as urban canyons, the use of multi-GNSS can reduce the negative impact of the poor satellite availability and allow coordinate accuracies to be more comparable to those presented in Table 1 and Table 2 despite the sub-optimal conditions. The precision improvements in these severe conditions can be up to 50% compared with much smaller improvements of 10-20% in optimal conditions (TSA 2015).

In PPP solutions the combination of GNSS systems can have a significant impact on the coordinate solution, especially the convergence time required to obtain desired levels of accuracy. The impact level on long observation periods (such as 24 hours) is lower with precision levels potentially improving by up to 30% (Li et al. 2015) when comparing GPS only solutions as given in Table 1 with a solution using all four global constellations. Multiple constellations use allows more noticeable improvements in shorter observation windows with improvements of 40-50% possible for 6 hour durations and ~60% improvements for 1 hour observation windows.

Improvements in overall precision or convergence times can also be seen when just using two constellations, but the improvement levels are not typically as large as when using all four constellations, with improvements of 20-50% depending on duration and satellite geometry (Li et al. 2015).

4 Survey errors

Survey errors exist in all measurements that are taken, with GNSS no exception. However, it is essential that surveyors recognise the potential size and impact of these errors on a project to ensure specifications can be met. The main error sources affecting the precision and accuracy of GNSS positioning are:

- satellite orbits (ephemeris)
- satellite clocks
- ionospheric refraction
- tropospheric refraction
- multipath
- interference
- coordinate transformations
- survey procedures.

Dependent on the GNSS survey methodology being implemented, each potential error source can have a varying level of impact on the position estimates. For example, in differential GNSS techniques the majority of atmospheric effect can cancel out over short distances whereas multipath errors are independent at each station being observed. Errors such as the coordinate transformations and procedural errors have the potential to cause the largest errors in the observations but should also be the easiest to avoid by following the appropriate methods.

4.1 Satellite orbits and clock

A crucial part of any GNSS survey computation is the location of the GNSS satellite and the accuracy of the GNSS satellite clocks. In real time the information about the current and future location of the satellites as well as values for the satellite clocks is obtained from the broadcast ephemeris transmitted by the satellites. However, the accuracy of these satellite orbit and clock estimates are approximately 100 cm and 5 ns respectively as shown in Table 3. For survey methodologies where observations are relative to a second receiver any errors in the satellite orbit and clock values are largely cancelled out and therefore do not pose as great a problem as in standalone positioning methods such as PPP.

To obtain higher levels of precision, international processing centres observe data from a series of global reference stations to compute improved satellite orbit and clock products. Use of these is essential in the standalone approach.

Depending on the accuracy and timing requirements of any applications, different orbit and clock products are offered. Table 3 outlines the different types of International GNSS Service

(IGS) products offered from ultra-rapid to final with improving levels of accuracy. The latency values state how long after data collection these products will be available for download and differ from a few hours to a couple of weeks

Satellite orbit and clock products are computed by multiple different organisations such as the (IGS) and the Center for Orbit Determination in Europe (CODE). Products can be downloaded free of charge, however it is essential that orbit and clock products are obtained from the same source and the products are not mixed.

Type		Accuracy	Latency
Broadcast	orbits	~100 cm	real time
	Sat. clocks	~5 ns RMS ~2.5 ns SDev	
Ultra-rapid (predicted half)	orbits	~5 cm	real time
	Sat. clocks	~3 ns RMS ~1.5 ns SDev	

Ultra-rapid (observed half)	orbits	~3 cm	3 – 9 hours
	Sat. clocks	~150 ps RMS ~50 ps SDev	
Rapid	orbits	~2.5 cm	17 – 41 hours
	Sat. & Stn. clocks	~75 ps RMS ~25 ps SDev	
Final	orbits	~2.5 cm	12 – 18 days
	Sat. & Stn. clocks	~75 ps RMS ~20 ps SDev	

Table 3: Orbit and clock products

4.2 Ionospheric refraction

The microwave signals sent from the GNSS satellites will pass through the ionosphere that lies in the Earth's atmosphere between around 50 km and 1000 km above the Earth's surface. The electrons in the ionosphere affect the transmission speed of the microwaves but this effect is different for GNSS code and phase measurements. The code measurements are delayed (i.e. the distances are measured longer than they really are), and the phase is advanced as the signals pass through the ionosphere. The ionosphere is also a dispersive medium at radio frequencies, and the effects will vary, depending on the frequency of the signals.

The other contributing factor to the variation is the total electron count (TEC) in the ionosphere. This factor is governed by the activity of the sun and the level of ionosphere changes with a number of well-known periodicities, including the 11-year sun spot cycle, a seasonal cycle and a diurnal cycle. Magnetic storms superimpose a sizeable irregular pattern over these cycles, making the prediction of the TEC very difficult. During a solar maximum, the vertical ionospheric error can reach up to 10 m (at GNSS frequencies) during the day, reducing to 1-2 m at night.

The frequency dependence of the ionospheric effect allows for a relatively straightforward elimination of the effect, provided observations are made at two frequencies. Hence, for high accuracy surveys dual frequency receivers are recommended, especially while observing longer baselines when using differential techniques.

The situation is less straightforward for users of single-frequency GNSS instruments. The variability in the state of the ionosphere makes it very difficult to produce a reliable estimate of the electron density and hence eliminate the effect. Although more complex models are available, the GNSS navigation message includes a simple model developed by Klobuchar. This model is based on a simple cosine representation of the ionospheric delay, and the amplitude and period of the model are allowed to change as a function of local time and geomagnetic latitude. A constant night-time delay is adopted.

This model represents something of a compromise between accuracy and computational complexity, such that the accuracy is limited to approximately 50% of the total effect. For relative positioning, either with code or carrier phase measurements, the differencing process tends to cancel the effects of the ionospheric delays. Provided the two receivers are reasonably close to each other (less than 20–30 km apart), it can be assumed that the signals are travelling through the same portion of the ionosphere, hence the effects on the received signals will be the same. By differencing the observations, either directly as in GNSS processing software, or indirectly as in DGNSS, this common error will be cancelled. As the separation between the two receivers increases, this assumption becomes less valid.

4.3 Tropospheric refraction

At radio frequencies the troposphere is a nondispersive medium. Thus, its effect cannot be eliminated from two-frequency measurements as with ionospheric refraction. The mitigation of tropospheric delay relies, instead, on the use of one of a number of models. These models generally characterise the troposphere as two components: a dry component and a wet component. Another, and usually more accurate method depending on the positioning mode, is the use of the GNSS data to estimate a tropospheric delay parameter.

The dry term accounts for about 90% of the total effect and can be accurately modelled from measurements of surface pressure alone (Hopfield, 1971). Errors do remain in these models due to the difficulty in assessing where the top of the troposphere is and horizontal gradients in the atmosphere.

The remaining contribution from the wet term is more difficult to model accurately since measurements of temperature and partial water vapour pressure at the antenna are generally not representative of the conditions along the signal path to the satellite. Partial water vapour pressure distribution in the atmosphere is extremely variable. Consequently, it is virtually impossible to model the horizontal and vertical gradients to a high degree of accuracy.

Table 4 gives a general indication of the errors attributable to both wet and dry terms at various elevation angles.

Elevation angle (°)	Dry term (m)	Wet term (m)
90	2.3	0.2
30	4.6	0.4
15	8.9	0.8
10	13.0	1.2

5	26.0	2.3

Table 4: Approximate tropospheric delay by elevation angle

Various methods are available for measuring the amount of water vapour in the atmosphere, although the expense and inconvenience of these methods mean that none is ideal from the point of view of a field user of GNSS.

The tropospheric effect can be mitigated either by using an empirical modelled or estimated as a parameter in the solution. The application of models should, in most cases, reduce the tropospheric error to a few cm, with the remaining uncertainty being attributable mainly to the wet component. Errors in the models become more significant at lower elevation angles, and it is common practice to restrict observations to a minimum elevation of, say, 15 degrees. In positioning modes such as long baseline RTK and PPP the troposphere delay should usually be estimated as a parameter alongside the station coordinates.

Errors in the troposphere that are common to two receivers will cancel in the same way as the ionospheric errors for relative positioning. However, the use of dual-frequency over long baselines will not lead to any improvement. Absolute positioning errors can be up to several metres if left uncorrected, although the application of one of the standard models should reduce the effect to the decimetre level, with errors in station height again being the most pronounced.

4.4 Multipath

Multipath in GNSS measurements is a propagation error caused by the reflection of GNSS signals en route between the transmitting satellite and the receiving antenna. Signals that have not followed the direct line of sight between the satellite and receiver interfere with the direct signals and cause an error in the measurements. Multipath can occur at both the transmitting and receiving end of the signal, i.e. at the satellite and at the ground antenna. The magnitude of the effect can reach several metres in pseudorange measurements, and errors of this kind directly affect the accuracy of a positioning solution. Fortunately, the effect on carrier phase measurements is about two orders of magnitude less than on the code measurements. Although carrier phase may be the primary observable for surveying and geodetic applications, the pseudorange measurements are used in order to resolve the initial ambiguities of the phase measurements.

The effect of multipath is different for each satellite and varies as the satellite moves across the sky. Since the effect is dependent on the location of the receiving antenna, multipath errors clearly cannot be eliminated by differential corrections or data differencing. In fact, as multipath at a mobile receiver and multipath at a reference receiver are uncorrelated, they may consequently have an additive effect.

The factors that determine whether a particular GNSS installation is susceptible to multipath are many and varied. The location of the GNSS antenna with respect to reflective surfaces, the design of the antenna, the type of correlator in the receiver, the signal to noise ratio, and the elevation and azimuth of the satellites, all contribute.

If an environment is 'unfriendly' in terms of multipath, the best strategy is to observe over the longest possible period of time, in an attempt to average out the effect. This has obvious implications for methods with short occupation times of the order of 15 minutes or less. For higher precision applications it is suggested that two occupations are observed separated by more than 20 minutes to observe a different satellite configuration. However, it is important that the reoccupation time is not the same time on a different day due to the sidereal effect of the multipath as discussed in section 4.4.1.

Antenna design is becoming increasingly advanced to cope with the effects of multipath, through the use of polarised antennas and choke rings. However, multipath remains a potentially large error in 'unfriendly' environments. If possible, it is far better to observe somewhere else and transfer a coordinate to the required location using a traditional survey method. Nevertheless, if the point is observed with GNSS, methodologies have been developed to investigate the levels of multipath at a given station as discussed in section 4.4.1.

4.4.1 Multipath determination

In applications where it has been decided that the levels of multipath at a station should be investigated a static test can be undertaken. In general, it can be assumed that for an antenna site which experiences multipath in carrier phase measurements, there will also be an effect on the pseudoranges, simply because the site has reflective surfaces in the vicinity. However, the effect on pseudoranges is not exactly the same as the effect on carrier phase and could be many times larger. One standard method of identifying carrier phase multipath with GPS observations is to examine residual plots on consecutive days. If trends can be seen from day to day, one can infer that the residuals are not due to random effects, such as the atmosphere, but will be due to multipath.

The premise behind this method of detecting multipath is that, for a static receiver, multipath effects repeat on successive days due to the orbital period of GPS satellites. Since the location of any reflective surfaces, relative to the antenna, does not change, signals from a satellite at a given location in the sky will always experience the same multipath. Since the orbits of the GPS satellites are such that each satellite will appear at the same point in the sky every 23 hours and 56 minutes (1 sidereal day), multipath effects for GPS observations on each satellite will repeat daily.

This test does have a number of drawbacks from an operational point of view. Since the baseline processing involves the use of the double difference carrier phase observable (involving two satellites and two receivers), it is not possible to identify which of the two satellites involved is contaminated by multipath, and more importantly, which of the two receivers is contaminated.

Thus, great care would have to be exercised in locating the second antenna away from any reflective surfaces, to ensure that the test could correctly isolate multipath effects in the antenna under investigation.

4.5 Interference

GNSS signals are particularly sensitive to interference from other transmissions, either as unintentional interference or from deliberate jamming. The sensitivity is due to the very low levels of the GNSS signals, so low that the resultant signals are below the ambient noise level at radio frequencies. The Standard Positioning Service (SPS) is considerably more sensitive to interference than the military Precise Positioning Service (PPS). Signals at sufficient levels to affect GNSS reception could arise from the upper harmonics of microwave links, radar, television transmitters, satellite and VHF communications and amateur radio. In addition, electrical noise from electronics and electrical machinery may also cause problems.

The problem of interference and jamming is being addressed by the receiver manufacturers, and an increased robustness to interference largely depends on a good front-end RF filter. Additionally, Controlled Radiation Pattern Array (CRPA) antennas have been designed to limit jamming. They do this by detecting the direction of the jamming and ignoring this part of the sky while boosting reception from remaining areas.

GNSS is so susceptible to interference because the radio frequency bands used are not generally protected. The impact of interference on the receiver can be quite varied, ranging from complete loss of lock on the signals to the introduction of cycle slips into carrier phase measurements.

Improvements in receiver design will not eliminate all problems and the simplest approach is to avoid sites of possible interference, particularly areas around radio and television transmitters. GNSS interference can also be very variable in nature and so it is important to build adequate redundancy into a network to cope with losses of data due to interference.

4.6 Antenna phase centres

Until the late 1990s, antenna phase centre variations were often not modelled as part of the GNSS processing techniques. Such variations of the electrical phase centre of a GNSS antenna are a function of the elevation and azimuth angle between the receiver/antenna and a satellite. These variations are particularly evident in the vertical direction and are dependent on the GNSS antenna type, GNSS constellation and signal frequency. If the same type of antenna is used on a relatively short baseline, the antennas’ phase centre variations will be cancelled out because both antennas have practically the same elevation angle to a particular satellite.

However, as baseline length increases, the elevation angles to a satellite from both ends of a baseline differ and the antenna phase centre variations become more significant. Furthermore, if different GNSS receiver/antenna types are being used, the different antenna phase centre variations will not cancel and must be accounted for. When using data from a CORS network it is important to try to use the same phase centre variations as used by the CORS operator when computing the CORS coordinates. Most CORS operators will publish the phase centre variations that apply to their stations.

The variations between different antennas can be up to a few cm therefore should be considered in higher precision applications. For RTK use, it is also important to understand which antenna is being used at the base, if this is different from the rover. Phase centre variations can be modelled relative to a well-defined standard antenna. However, since 2006 the IGS standard has been to use absolute antenna phase centre corrections. These corrections are determined using a robotic system and include azimuthal values and elevation down to 0 degrees. These antenna phase centre corrections are available for the majority of antenna types and provided by the IGS in a standard Antenna Exchange Format (ANTEX). These values can be inputted in most GNSS processing software to correct for the antenna phase centre variations.

4.7 Vegetation cover

GNSS has been shown to provide position estimates still under vegetation cover, such as a tree canopy. However, there may be a significant loss of signal and a corresponding increase in the signal/noise ratio (SNR). This is particularly dependent on the moisture content of the vegetation, and on the type of vegetation. The higher the water content, the greater the signal loss. At the limit this may result in complete signal loss, which will cause the loss of satellites from the position solution, particularly in dynamic surveys when the GNSS equipment is moving to capture the survey data.

In these cases, it is far better to establish tertiary or secondary control using a rapid-static survey method at the edges of the vegetation where there is a clear view of the sky. Further control can then be added under the vegetation canopy using a traditional traverse and spirit levelling method. Detail survey observations should also be carried out under the vegetation canopy using a traditional method, as RTK methods are not satisfactory in these marginal conditions.

5 Coordinate reference systems

In this section a description of coordinate reference systems and transformation related to GNSS positioning can be found. Further detailed information on coordinate systems within Great Britain can be found in the Guide to Coordinate systems in Great Britain by Ordnance Survey. These basic principles can be applied to other countries across the world but further information on local mapping systems should be sought before conducting any work.

Coordinate systems are an essential part of any survey, especially when GNSS is being used. Points on the Earth's surface measured by GNSS can be defined in a global reference frame in terms of latitude, longitude and ellipsoidal height or in a three-dimensional Cartesian system (XYZ) with the origin at the Earth's centre of mass. Both of these coordinate systems can subsequently be converted into easting and northing projection coordinates, although Cartesian XYZ coordinates would first have to be converted to geodetic latitude, longitude and height.

5.1 The geoid

The geoid is the equipotential surface (a surface where gravitational potential is constant) of the Earth's gravity field that best fits global mean sea level. Heights referred to sea level are therefore related to the geoid and are termed orthometric heights. Sea level changes with time, and so most countries define the height of one point with respect to mean sea level over some time period and refer to this defined point as the levelling datum. Positions determined by geodetic astronomy are related to the geoid in as much as the axes of the instrument being used are aligned in the direction of, and at right angles to, the direction of gravity.

5.2 The ellipsoid

The mathematics of computing the geoid are complicated, and various approximations to its shape have been made. As a first attempt, a spherical Earth introduces negligible error for some cartographic purposes; the attraction of this choice is that it is a surface with constant curvature. However, an ellipse of rotation, with the semi-minor axis in the polar plane and the semi-major axis in the equatorial plane, is a better match to the geoid, although the variation can reach ±70 m. This shape (the ellipsoid) has been used for geodetic purposes for over two hundred years. A variety of 'best-fitting' ellipsoids (mainly national, regional or global), each with a different size and orientation to the Earth's spin axis, have been used.

5.3 Datums

Each GNSS system has its own datum: for example, GPS uses WGS84. A datum in this context is essentially a set of conventions, constraints and formulae that define it without recourse to a physical infrastructure. In order to use this datum practically it must be physically realised. This is achieved by means of a known network of points with defined coordinates, known as a terrestrial reference frame (TRF). Each GNSS constellation has its own TRF and these reference frames are updated every few years with the frame name indicating the date at which observations are used up to, such as ITRF14. What datum used by the surveyor will depend on the particular job requirements, e.g. national datums are not necessarily based on the latest ITRF but may still be GNSS-compatible, e.g. ETRS89. Thereafter transformations will also often be applied to express the position in the national mapping datum.

5.4 Map projections

A projection is required to transfer measurements and positions from the ellipsoid onto a flat surface suitable for making into a map. For example, in Great Britain, the Ordnance Survey uses a modified version of the Transverse Mercator projection. In a simple Transverse Mercator projection the surface of the local ellipsoid chosen to represent the Earth is represented on a cylinder that touches this ellipsoid along a chosen meridian and which is then unwrapped. The scale is therefore correct along this central meridian and increases on either side of it.

In the example of Great Britain, the modification made to the projection is to reduce the scale on the central meridian by a factor of approximately 0.9996. The projection then becomes correct in scale along two lines nearly parallel with and on either side of the central meridian about two-thirds of the way between it and the edges of the projection. On the edges, the projection scale will have increased to approximately 1.0004 of nominal figure. The advantage of this modification is that it extends greatly the area covered by the projection within a minimised distortion of scale.

This is shown in Figure 2.

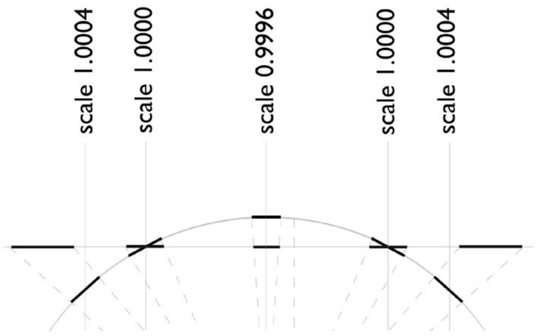

Figure 2: Scale factor

The horizontal line represents the map with points at true scale where it intersects the arc. The central area has a scale of less than one, the outer area scale is increasing up to maximum of 1.0004.

For high accuracy applications such as infrastructure engineering, it is likely that a national mapping grid is not optimal due to relatively high distortion characteristics. In such cases a focused low distortion projection can be more appropriate. An example of this is the London Survey Grid, a customised Transverse Mercator projection used for both underground and overground engineering works.

5.5 Projection coordinates

For most general survey and mapping projects, projection coordinates are the most useful coordinates. The surface of the Earth is represented by a grid of eastings and northings and mapped features, structures or other items all have specific grid coordinates. Simple numerical grid computations can be made between any of the mapped items such as distances, areas or volumes, hence these are used in almost all engineering and mapping applications, although map scale factors also need to be considered when using these coordinates.

The transformation between the GNSS coordinates and the projected grid coordinates is carried out using a set of mathematical formulae applied within GNSS receiver systems and processing software. Due to small variations in numerical constants, coordinate transformation can vary slightly between manufacturers. It is therefore important on high accuracy projects that the transformation method is kept consistent throughout. Consequently, it is critical in any project to document what transformations have been undertaken, the software used including version number and finally any variable parameters used.

5.6 Snake projection

Large projects with sites that extend above a few kilometres can have large scale factor distortions. The choice of projection(s) can have a large impact on the size of the distortions, especially on largely linear sites. The snake projection is a continuous near zero grid coordinate system for sites that can extend hundreds of kilometres. This type of projection can minimise scale factor distortions to less than 20 parts per million (ppm) a few kilometres either side of the selected projection route. With errors less than 20 ppm the projection can be assumed to have zero distortion for almost all project specifications.

Control coordinates taken with GNSS observations can be converted using relevant software into the snake projection defined for the project. Once control observations have been projected, further work such as traverse observations do not need to be corrected for scale factor distortions so long as the points are along the corridor of the snake projection.

5.7 Height transformations

There is a special treatment required to calculate the geoid or mean sea level heights of points observed by GNSS. A 'simple' transformation usually does not apply and a geoid model has to be used.

The orthometric height of a control point is the height above the geoid in that area, usually given the symbol *H.* The GNSS height, above the ellipsoid, of the point is usually given the symbol *h*. The difference between the two heights is the height of the geoid above (or below) the ellipsoid, or geoid-local ellipsoid separation, *N*. This is shown in Figure 3 with an exaggerated separation for clarity. A geoid model maps the changing values of *N* over an area.

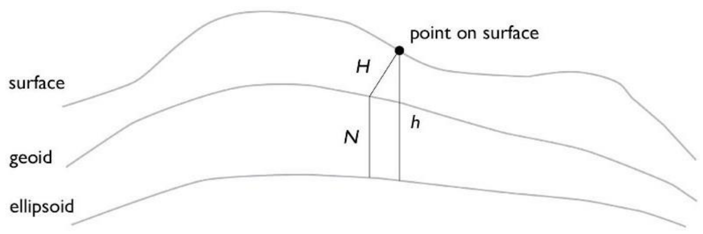

Figure 3: Height relationships

Thus, the general equation to relate heights can be written as: $H = h - N$

With a global geocentric ellipsoid used with GNSS positioning, such as International Earth Rotation and Reference Systems Service (IERS), the separation *N* can vary dramatically. For example, in Great Britain the geoidal height is approximately 50m, while in areas of the Middle East it is -10m, i.e. the ellipsoid is 10m above the geoid. Thus, height corrections for the geoid/ellipsoid separation are extremely important when transforming ITRS geodetic latitude, longitude and ellipsoidal heights to coordinates with a local orthometric height.

There are many different geoid models published and these provide the basic look-up table for orthometric/ellipsoid separation values over a given area. Both global and national models exist, and most give the values between the global or local geoid and the IERS ellipsoid. The models are incorporated differently into GNSS manufacturers' software, and it is good practice to check which model, and associated accuracy, is being used.

GNSS observed heights are converted from Cartesian Earth-centred Earth-fixed (ECEF) to ellipsoidal coordinates and, by an application of the orthometric/ellipsoid separation from a geoid model, the resultant orthometric heights are produced.

Depending on the local situation, best practice differs:

- use GNSS plus the orthometric/ellipsoid separation model to bring orthometric height onto a site. Traditional spirit levelling should then be used to promulgate the orthometric height around the site
- include at least three or four height control points in a survey network which have reliable orthometric heights.

These orthometric heights can then be compared with the computed values from the national control and the application of the geoid model. The figures may show up a bias in the model, especially if it is too large a grid global geoid model. They can be fixed in any final adjustment if required.

5.8 Coordinate transformation issues in GNSS

In GNSS land surveys, there are several key coordinate system and transformation issues that should be considered.

Firstly, there is the use of national control points. For any project a minimum of three (preferably four) points should be obtained to control the survey network. An indication of the actual accuracy of these points should also be obtained from the national mapping agency, as older control values from triangulation surveys will not be as accurate as GNSS values. If these older 'fundamental coordinates' are held fixed in any network adjustment, the GNSS survey geometry will be distorted and will result in errors.

Therefore, good quality, recently surveyed national control points should be obtained for any survey that requires a tie to a national grid.

Secondly, if no national grid coordinates are required and no national control is used for a survey, it is important to use an accurate position in ITRS as the basis for the survey. If this is not done, scale errors will result in the baselines observed from this base point. The approximate magnitudes are shown below in Table 5.

Error in true ITRS position (m)	Baseline scale error (ppm)	Approx. error at 5km
2m	0.3	2mm
10m	2.6	13mm
100m	25.5	130mm

Table 5: Scale error magnitudes

Thirdly is the use of manufacturers' own single-step transformations. These are often offered by GNSS equipment manufacturers as a simple method of obtaining true scale (flat earth) local grid coordinates from ITRS. In the worst case, an arbitrary ITRS point is used with the manufacturer's own single-step transformation for the whole survey. If future work is to be carried out using another manufacturer's equipment, many potential problems could arise. It is far better to conduct the control element of the survey separately, based on precise ITRS points and using rigorous transformations from ITRS to the final local grid. It is best practice to use single-step transformations generated from a set of four or more points that have accurately surveyed coordinates in ITRS and in the local grid. It is critical that detail observations are completely within the control framework when using single step transformations.

Many regions have 'official' national transformation models to relate GNSS positions to the national mapping datum. These are not always single-step simple transformations and can be complex e.g. rubber-sheet type models to account for variations between GNSS datums and legacy mapping datums. The use of official transformation models (if they are available) is often preferable to simple single step models from manufacturers.

If a manufacturer's single-step or similar transformation is used, all parameters and transformation types should be quoted in any coordinate listing or in the survey report, along with equipment and software versions. A back-up copy of the survey should also be kept in ITRS for any other later transformations as required.

6 Quality control

Within a procedure or specification, aspects of quality control should be considered, i.e. ways of ensuring that care is taken during a survey to achieve the best possible result. These quality control procedures should ensure the survey is executed in the most effective way, with the minimum opportunity for errors to occur. To achieve this in practice, it is recommended that the planning process should include a simple, straightforward quality plan, which will reference the necessary practical procedures. This will explain what will be done during the execution of the survey to assure both the client and surveyor of the controls being exercised over quality.

No generic plan can be written to cover all GNSS surveys, due to the diversity of the tasks, methods and personnel who perform the works. The quality plan could be as simple as a single page of basic quality control procedures, or a complete manual for a large project. It should cover:

- how quality control of the field work and field data will be achieved
- how quality control of the office procedures will be achieved
- any QA management systems and procedures of the survey company
- any work instructions of the company (including health and safety procedures).

Field work aspects should include items relating to data download and back-up from receivers, staffing and logistic requirements, storage of data, preprocessing of data, transport and overnight storage of equipment and supply of back-up equipment. Office procedures would include some of these and other relevant aspects, such as data back-up and archive.

It is essential that all aspects of the survey and data handling process are considered at the stage of producing the quality plan, since it will not always be immediately apparent in the subsequent fieldwork that inadequate observations have been obtained. A considered redundancy of data is always preferable to a shortfall that requires reobservation to obtain a solution to the required quality.

6.1 Network design

Despite the perception that GNSS is a high-tech activity, the basic rules of surveying still apply to GNSS networks. The principles of robustness, overall cover and repeatability apply. The latter aspect is often overlooked.

In a GNSS survey, trivial baselines exist in an observing session. For example, if three GNSS receivers are observing at the same time in a static control survey, there are three GNSS baselines that can be processed. Two of these are independent,however, the third, computed from the same observations, is just the vector difference of the other two. This is known as a trivial baseline, and it is best practice not to compute this baseline. The processing and inclusion of trivial baselines will give highly correlated measurements in the network adjustment. This will cause errors in the stochastic modelling, and incorrect results in the adjustment. For trivial vectors it is better to have a second setup, during another session and perhaps on a different day, or to use a different time interval of the same session.

It should rarely be assumed that a survey is to be a once-only, standalone activity, unless the specification makes this quite clear. The normal assumption should be that further observations and activities, not always undertaken by GNSS methods, will take place over an extended period of time. Adequate permanent reference points should be located, with due consideration to any future construction works, so as to facilitate direct linkage with later surveys and ensure that reference points are unlikely to be destroyed by construction works (at least until the latter stages of the project). In particular, the situation may arise where there is some variation between two networks that are unconnected but adjacent. This could be due to different fundamental control being used for each. When designing a new survey, it is good practice to ensure that some common network points exist in the two adjacent survey areas, to enable correlation and computation of coordinate differences.

6.2 Network shape

Networks should be designed to completely encompass the area of the survey detail, whenever access permits. Within the network areas an individual point will be constrained by the residuals of the network stations, but observations outside the enclosed figure will have little constraint and a rapid increase in error can occur, even over short distances.

Closed loops provide the best geometrical figures, more sessions observed for each baseline, generally leading to higher precision coordinates. For highest accuracy networks, the repeating of one or more previous baseline sessions in the next group to be observed provides additional robustness. Radial baselines in a network should be avoided for main figures, as these do not provide independent checks between the outer points. Although multiple radial set-ups would provide such checks, better geometry, statistical certainty and economy of operation will be achieved by use of traditional closed loops or figures similar to older triangulation chains.

6.3 Linkage to a national CORS network

When connecting survey areas to national control, it is better to design this part of the network separately where appropriate. In some cases, long lines may be needed to connect and tie into national control, especially if the data is from an active system where the three nearest control stations could be up to 100 km distant. In these cases, it is far better to have a ‘zero order’ network to connect three or four of the primary control points to the national control. This will ensure the baselines in the network are of a similar type and will improve the stochastic modelling in a network adjustment. These primary points can then be fixed in this way with their associated precision values and used for the 'primary survey'. This is particularly important when combining short baselines (e.g. 1 to 3 km length) with baselines that connect to national control of between 50 and 100 km.

It is not good practice to observe as many points as possible, put all the data into the processing software and use it as an automatic machine. Far better is a rigorous step-by-step approach of loading vectors and processing from the whole to the part, just as in traditional surveys.

6.4 Quality control of real-time systems

Real-time systems are a highly productive GNSS technique, in that the surveyor knows the system has worked and that sufficient data has been logged before vacating the site. The surveyor knows immediately that the data has been successfully captured at a suitable level of precision, although the absolute accuracy is unknown.

As no post-processing and adjustment is available to provide statistical analysis, the quality control parameters for these surveys should be available in real time to the surveyor.

The surveyor should view, in real time, precision figures determining solution quality to check the impact of any multipath problems or accuracy degradation at a point and should set tolerance values into the datalogger making it impossible to record sub-standard data.

In post-processing, multipath would only be detected by a point failure or inability to initialise. Longer-range, high-accuracy, real-time systems are also offered by some manufacturers. These introduce further issues with respect to coordinate control, as well as initialisation strategy and reliability.

In all cases when using network RTK the best practice guidance notes for Network RTK Surveying in Great Britain and the manufacturer's guidelines should be followed. In particular, it is good practice to initialise the survey twice when using either real time or post-processed network RTK. For example, an on-the-fly initialisation could be made first, followed by a second initialisation on a known point. This ensures a check on the initialisation is made at the start of the survey. In addition, further check measurements should be made on previously surveyed points that are part of a larger primary control network, and these check measurements should be used to assess the absolute survey accuracy.

Real-time dynamic systems produced by manufacturers may well offer ad-hoc indicators of quality and reliability. The quality control can be split into two parts:

- system initialisation and subsequent initialisations after a loss of lock
- quality control of the coordinates produced in real time.

For quality control of the coordinates produced in real time, the testing of integer solution sets in the manufacturer's software should follow a similar line to that used in the post-processing systems. Outlier detection should still be implicit, based on testing individual double difference phase residuals against a scaled RMS of all residuals. Variance ratio testing should be carried out by computing the ratio of the next best to the best solution variance and comparing it against a value computed using the Fisher distribution at 95% confidence. More conservative ratio test values of 5.0 or above are usually used for real-time RTK systems.

6.4.1 Reporting of quality control

Much of this quality control will take place in the manufacturer's software and will not be visible to the surveyor. It is recommended that the following quality control parameters are produced for all detail points to give an indication of coordinate precision to the surveyor:

- the number of satellites being observed
- the *a posteriori* standard errors of the baseline solution (dx, dy, dz)
- 95% confidence level error ellipse, as horizontal and vertical precision figures.

The final output should present these quality control parameters with the coordinates of each point. Similar information should also be displayed on screen to the operator for every point observed and any points that fail should not be logged. This implies that the operator checks the screen for every point recorded during the survey; a more realistic option is to set up an acceptance value in the logger software.

6.4.2 Control point inclusion

In all real-time kinematic surveys, it is vital that the site includes at least one high-accuracy and precision control point previously established using static GNSS or similar technique. This allows real-time coordinates to be checked against a known reference. It also provides a convenient reinitialisation point in the site area. If the real-time solutions are to be later included to strengthen a network adjustment, or as tertiary control points, they should also include as a minimum the upper triangular elements of the *a posteriori* covariance matrix. The RTK observed control points should also have a longer occupation period than the detail points, perhaps two or three minutes per point.

6.4.3 System initialisation

Operational procedures are generally simple; with a dual-frequency receiver, initialisation is fully automatic and can be realised while static or while moving (on-the-fly). Fully automatic initialisation, under normal conditions, is usually achieved reliably within a minute or so. To achieve cm accuracies, the rover tracks carrier phase and code phase (pseudorange) data and should initialise itself with respect to the base station. Initialisation is the term used to define the process of determining the carrier phase integer ambiguities; at least five common satellites should be tracked between the base station and rover with a PDOP of better than five. Before initialisation, the receiver may produce a coarser solution, within about a metre, but can converge within a minute or so to a few cm. Once initialised, the system switches to a fixed solution (ambiguities resolved) and precision improves to about a cm.

6.4.4 Incorrect initialisations

The initialisation process is very reliable, but incorrect initialisations can occur. Formal testing by a manufacturer should indicate initialisation reliability figures of at least 99.75%. The level of reliability may also be a function of time, with less reliable initialisation figures being specified for shorter initialisation periods. Different systems respond in various ways, and this should be taken into consideration when preparing quality procedures. In all cases, best practice is to initialise in completely open areas with no possibility of multipath or other interference to satellite tracking.

For RTK initialisations, the production of three distinct quality control parameters is recommended:

- the ITRS coordinates of each initialisation point, their accuracy at the 95% confidence level and the type of initialisation (i.e. known point, on-the-fly, etc.)
- integer bias values of the initialisation baseline solution
- ambiguity ratio and RMS of the residuals.

The latter two items should be provided by the software to give the surveyor an indication of the reliability of each initialisation and should be checked after each reinitialisation with a duplicate field point taken to provide a physical check. All three should be logged as explicit quality control parameters in the survey archive for each initialisation and given in the final report if required.

6.4.5 Initialisation checks

A bad initialisation may result in position errors of at least 20 cm. The receiver may automatically detect and correct these events, given suitable satellite availability and geometry. This would normally take a few minutes, and this time could be considered as an initialisation or self-checking window. The rare occurrences of bad initialisations tend to be focused around lack of redundant satellite coverage or poor PDOP (e.g. five satellites only with a marginal PDOP), or at heavily multipathed locations. Operational procedures, such as satellite coverage planning or checking against known points, are best practice to negate such cases.

Similarly, observation periods where an initialisation has been followed quickly by a loss of lock and re-initialisation (i.e. within the time of a self-checking window) should be treated as suspect, and two checks should be done at known points in this segment.

A fixed solution, while requiring five satellites to initialise, can be maintained through periods when only tracking four satellites. In addition, the fixed solution can be maintained through periods when the communications link is lost. The communications link provides satellite observations from the base station to the rover, in order that the firmware can compute the initialisation baseline solution. It is best practice to note when the communications link is lost, and to reprocess any points captured, using post-processing software. In most cases the RTK equipment will flag the observations, but it is good practice to log these points on a suitable booking sheet in the field. If satellite lock is also lost when the communications link is down, it is good practice to complete a period of static initialisation according to the static positioning guidelines given in section 3.1.

It is best practice, however, to ensure that the communications link is maintained for initialisations and survey points. This can be done by planning the survey to ensure RTK baselines are kept short, and the location has a minimum interference. A radio frequency scanner can be taken to marginal or unknown sites. This will aid selection of an optimum communications frequency. A repeater radio station could also be strategically located to ensure communication links are maintained.

6.4.6 Real-time DGNSS

When using these systems for land surveys, it is good practice to ensure that more than one epoch of data is used at a point. Epoch settings of typically one second should be used, with five to ten readings taken per point, according to the requirements of the specification. This will allow the computation of the following statistical quality control measures for each point:

- number of satellites
- PDOP
- standard deviation of positions logged
- overall software estimated horizontal and vertical precision.

Other aspects of quality control, the derivation of specific quality control parameters and statistical tests for models used in DGNSS systems are well covered in Guidance on satellite-based positioning systems for offshore applications.

6.5 Recording of field notes

Booking sheets are vital to ensure the correct recording of antenna height and orientation, point name and data file name for the occupation. These are the most important factors to record in any GNSS survey. In addition, it is good practice to record the following additional items to assist in the data processing stage of the survey:

- date
- time of occupation

- epoch setting
- elevation mask
- equipment serial numbers (for fault tracing)
- weather observations
- obstructions to the view of the sky, and
- any other relevant observations.

The sheet can also be used to record times when cycle slips are observed to occur and when loss of lock or a new initialisation occurs in real-time surveys.

A recent almanac should be used with appropriate planning software to generate a sky plot of satellite coverage during the survey. This will clearly show the times during the day when satellite coverage is better than at others. It can also be used to plan the occupations for the day, or to plan a break in the survey when satellite coverage is poor. It is best practice for the surveyor to take a plot on site, showing the predicted number of satellites and expected PDOP or GDOP values.

6.6 Office procedures

As part of the overall quality system, the processes involved in converting raw field data to useable coordinate information needs to be considered. These include downloading and preliminary processing, full processing and key checks, adjustments and transformation to the final coordinate system.

6.6.1 Data handling and control

Downloaded raw data should always be fully backed up as an archive before any manipulation commences. Whenever possible, a preliminary batch process of data should be carried out immediately after completion of observations on site. This provides warning of possible processing problems while there is still an opportunity to carry out immediate reobservation of doubtful points. All such processing should be deleted after the data is confirmed to be adequate, to avoid it being treated as the final product. Full reprocessing with all controls applied can then take place and be used to supply the final data, as described below.

As part of the backing-up of information, consideration should be given to providing a RINEX format copy, since this is software independent.

6.6.2 Data processing software

For the raw data observed in static surveys and dynamic (non-real-time) surveys, described in section 3, post-processing is required to obtain final coordinates. It is recommended that wherever possible the software used is the current version offered by the GNSS hardware manufacturer whose equipment is used for the survey.

This will solve numerous incompatibility problems and also provide higher-precision coordinates, since a manufacturer often has specific codes stamped on the binary data to aid the post-processing. If this is not possible, it is recommended that data in RINEX format is used with the proposed software.

Most of the software used for baseline processing, loop closures, network adjustment and coordinate transformations has become increasingly automated. The result is that while more people can now successfully process GNSS data and use it practically for surveying tasks, fewer understand the processes which have to be applied to GNSS raw data. Advanced processing strategies are available in some packages, but these should only be used by experienced personnel with substantial expertise, since incorrect application of such techniques can introduce significant errors that are difficult to detect.

6.6.3 Quality of data

The important general elements of controlling the quality of data in post-processing are:

- ensuring the correct baseline solutions are processed and selected
- baseline observations outside the required precision are deleted or reobserved
- network adjustment is computed with a correct weighting strategy (stochastic model)
- coordinate transformations and projections are computed and applied correctly.

Each of these important aspects is covered separately in the following sections.

When attempting to obtain the best baseline solution from a set of possible options, the process of statistical testing is used in the manufacturer’s software. Such testing falls into two distinct categories:

- measurement quality estimation during GNSS processing (RMS and standard deviation)
- the testing of integer solution sets (ratio).

6.6.4 Mathematics of processing

The fundamental measurement used by static and dynamic GNSS surveying systems for baseline estimation is the double difference phase observable during the final pass of processing. The fundamental observables for the final pass are usually one of the following:

- L1 fixed double difference
- L2 fixed double difference
- ionosphere free (L3) double difference
- narrow lane double difference
- wide lane double difference.

GNSS processing software generally computes a root mean square (RMS) of the residuals for the chosen final pass double difference phase observable. This is then often scaled by a default factor (for example, 3), which corresponds to a number of standard deviations. Residuals computed from a least squares estimation are then compared with their scaled

RMS values and, if they exceed it, are removed from any further estimation process. The alternative to specifying a fixed scaling factor is to compute a value based on the Student's t-distribution at a specified confidence level (e.g. 95%). This value is based on the degrees of freedom in the measurement process and the specified confidence level.

This then provides the following as an estimation of the measurement quality: RMS, in cycles or metres, and the standard deviation of the baseline components dx, dy and dz at the 95% level. Error ellipses may also be computed with respect to the estimated coordinate solution.

Baseline differences between iterative solution types for the same baseline, such as wide lane and narrow lane combination and L1 fixed integer or ionospheric-free fixed integer should also be inspected for variations larger than the expected measurement precision. This is a key check and essential for all surveys.

For fixed integer solutions the testing of the different possible integer solutions is carried out. This is done by computing the ratio of the next smallest solution, to the smallest variance from the two integer solutions. There is usually a cut-off value, normally set at 1.5 for static baselines and 3.0 for post-processed kinematic baselines. The alternative to specifying a ratio cut-off value is to compute a value based on the Fisher distribution at a specified confidence level (e.g. 95%). The 1.5 value corresponds to the 95% confidence level. Thus, a ratio is also usually given for each fixed integer baseline.

6.6.5 Key parameters and checks

Many manufacturers have their own method of selecting the best baseline solutions from a set of available solutions. In addition, it is good practice to determine the manufacturer's range of values for ratio and RMS that are acceptable, as they will not always be computed in the same way. In many software packages the baseline selection will all be an automated procedure.

It is recommended that the following information supplied by the software is extracted from the baseline summary and quoted in a baseline processing section of a final survey report as three distinct QC parameters for each static or dynamic baseline solution accepted:

- RMS
- standard deviation of the baseline components
- ratio, for fixed integer baselines.

The manufacturer, type and version number of the baseline processing software should also be detailed in the survey report.

In control surveys, once baselines are successfully estimated and selected, loop closures should be computed around the geometric figure of the network as a further quality control check. This is a second key check to ensure baseline estimation has been correctly achieved. This check is of paramount importance when processing large numbers of baselines in batch jobs, as careful treatment of each baseline is unlikely to have been carried out. As a general rule, for closures of lengths between 10 and 50 km, those exceeding 3 ppm should be investigated. It should be noted that any misclosures left in the project will be propagated into the final coordinate error ellipses. Thus, if a specification requires final error ellipses to be better than 0.1m at 95% confidence, all misclosures of greater than 0.1 m should be eliminated to achieve the specification.

Where a new single point has been observed with reference to a number of base stations, the 'coordinate recovery' technique should be used. The source control point nearest the new point is treated as base station to compute the new point. Then, from the new point, baselines are computed out to the other source control stations. The difference between the computed and published coordinates will give an indication of the accuracy of the coordinates, highlighting dubious source control GNSS data.

6.6.6 Network adjustment

Following the GNSS baseline estimation in the baseline processing part of the software, GNSS vectors are generally passed to the network adjustment part of the software. GNSS vectors and the associated full covariance information should be passed across. However, it should be noted that for float solutions, where the baselines are still biased by the non-fixed integers, the covariant elements may be discarded prior to network adjustment.

In the network adjustment software there should be a capability to allow for different classes, or groups. This will allow different weighting strategies to be adopted for different classes of baseline solutions, or for different types of terrestrial observations. A combination of kinematic, rapid/fast static or static baselines can then be included, with properly ascribed significance, in a common adjustment. The combination of terrestrial observations with GNSS data is a complex field of study, and appropriate specialist combined network adjustment software should always be used by experienced personnel with substantial expertise if this is to be undertaken.

6.6.7 Network statistical testing and measurement weighting

Within a network adjustment the initial focus should be an 'inner constrained' network, based around testing of the individual measurements. A procedure following the computation of standardised residuals and their comparison against a chi-squared probability test (see Adjustment Computations: Spatial Data Analysis, 6th Edition) and the tau criterion is recommended, and should be included in any software. This may well take place as part of a preliminary, or free, adjustment. Within this process it is recommended that only non-trivial baselines are used. This avoids the problems of mathematically modelling correlated measurements.

Failure of testing against the chi-squared probability test and comparison against the tau criterion may be either representative of inadequate covariant information from the GNSS processing stage, network geometry limitations or true measurement deficiencies. Failure of testing should promote more evaluation and testing of the network before finally deciding upon explicit rejection of measurements.

Once the final set of measurements have been tested and selected, the adjustment should proceed with the appropriate weighting strategy applied. This can be a 'minimally constrained' network where user-defined minimum constraints are used and the corrections to the point coordinates are smaller, or a 'constrained' network where all or more than the minimum required control points are kept fixed, which can result in larger corrections to the point coordinates. This is a vital aspect of any network adjustment as the final output of the station error ellipses are dependent on the weighting strategy used. For example, centring and height errors should be entered. In addition, the stations selected as control should be held fixed at their defined values with their computed precision. It is best practice to carry out network adjustments in ITRS before any coordinate transformations are made.

6.6.8 Key parameters for reporting

There will be a host of statistical tests and quality control parameters presented by the software throughout the adjustment process: standard deviations of adjusted coordinates, station error ellipses, RMS figures, covariant values, histograms and observation residuals. From these, it is recommended that the most important quality control parameters extracted are the station error ellipses at the 95% confidence level (check the sigma values given by the software) and the weighting strategy applied. These should be presented in the report in the following ways.

For each adjustment:

- the weighting strategy adopted including the a priori standard errors used
- the overall unit variance for the network (which should be approximately equal to one).

For each station (given in the ITRS coordinate system):

- latitude and the 95% confidence value
- longitude and the 95% confidence value
- ellipsoid height and the 95% confidence value.

The manufacturer, type and version number of the network adjustment software should also be given in the final survey report.

6.6.9 Post-processed DGNSS surveys

The quality control measures outlined above in section 6.4.6 for real-time DGNSS surveys should also be computed for any post-processed surveys. It is recommended that if post-processed and real-time data are to be combined in a particular project, some common check points are taken in both the real-time as well as post-processed modes.

6.6.10 Coordinate transformations and projections

Transformation of the coordinates from the network adjustment can also be carried out almost automatically within the same software suite or it may be undertaken in a completely different manufacturers' or other software package. There are no real statistical confidence values that can be determined from transformation of coordinates, as often the transformation is a simple mathematical function; from one ellipsoid definition to another, and then to a defined projection and local grid. Therefore, when transforming GNSS coordinates from the network adjustment, the following quality control parameters should be supplied in the survey report:

- transformation technique used (e.g. Molodensky, Helmert, MRE, NTv2 OSTN02 – NTv2 format – see section 5.8)
- numerical definition of the new and ITRS ellipsoids
- actual values of transformation parameters used
- source of transformation parameters, e.g. NIMA, self-generated, national mapping agency
- accuracy of transformation parameters (see section 5)
- projection technique used and the numeric definition of the projection
- manufacturer, type and version of software
- new coordinates and final accuracy values in local datum or projection.

6.6.11 Data management

All recorded data should be compatible with National Spatial Data Infrastructures (NSDIs). New coordinates and final accuracy values in local datum or projection should be managed and maintained by surveyors in support of NSDI initiatives.

Appendix A: Sample specification with specific GNSS clause

The following could be part of a sample survey specification, based on RICS' Measured surveys of land, buildings and utilities.

The elements that are related to GNSS are detailed:

Control network

The surveyor will establish plan control at a density sufficient to achieve the specified accuracies. The main survey stations will be of stable construction. The surveyor should choose the most appropriate marker for each location.

The final survey grid will be arbitrary but related to the national grid. A description of the grid system used shall be quoted on each survey plan or upon the index plan.

The control stations will also be connected to the national grid values. The maximum error ellipse at 95% will not exceed (xx) mm throughout the main control network. For this survey GNSS coordinates are required for the control stations and static or fast static GNSS survey will be used for the control network.

The survey report will include copies of the control station sheets and a method statement describing the survey technique used. Copies of the final network adjustment report shall also be included.

Level network

The surveyor will establish vertical control at a density sufficient to achieve the specified accuracy of less than (yy) mm. The main site control station may be linked to the national vertical datum using a static GNSS control survey. A principal control station will also be levelled to a national control bench mark in good standing and the closure value stated.

Appendix B: Sample procedures relating to a GNSS survey

These sample procedures are based on a simple method statement that could be given for a basic GNSS survey of an open site area, for example a road bypass site, where GNSS might be used for the whole work.

Method statement – GNSS survey

1. Site work

1.1 Further to a site reconnaissance any existing survey control will be inspected for stability and marking.

1.2 New control stations shall be installed, marked and prepared on station description sheets in accordance with the specification. They shall be selected in GNSS-friendly location – where there exists a clear view of the sky to minimise cycle slips, multipath and other interference. Markers shall be road nails or earth anchors, as appropriate to the surface in which they are installed.

1.3 The following equipment shall be used for this survey … (state manufacturer, model/type)

1.4 One GNSS base station shall be established at a central location in the survey area. This shall be in an area with an open view of the sky as described above in paragraph 1.2.

1.5 As required, national control station(s) shall be observed or downloaded (via RINEX) to connect the survey to the national grid system.

1.6 Roving GNSS receivers will be established at other control stations forming a network around the whole site. These shall log data in a static mode at a rate and duration to achieve the accuracy required.

1.7 For the detail survey Real Time Kinematic (RTK) techniques shall be used within the above control network.

2. Office work

2.1 All GNSS survey data will be downloaded to office computers.

2.2 Survey data processing shall be carried out in … (state name of manufacturer's software).

2.3 Prior to processing the control network, antenna heights shall be checked twice, once on import and once during set up of processing parameters. Baselines shall be processed and checked, loop closures shall be computed and a network adjustment shall be performed. Coordinates shall be output in the specified grid system with their precision values, together with a listing containing the full details of the transformation and parameters used.

Band	Plan accuracy (X, Y)	Height accuracy (Z)	Example GNSS survey types/use	Legacy output scale	Min size of feature shown to scale (not symbolised)
A	3mm	5mm	Monitoring, high accuracy engineering setting out and fabrication surveys	1:5	5mm
B	5mm	5mm	Monitoring, high accuracy engineering and measured building surveys and setting out, static baseline	1:10	10mm
C	10mm	10mm	Engineering surveying and setting out, high accuracy measured building surveying, heritage recording, static baseline, rapid static, Static Network RTK, PPP	1:20	15mm
D	+/- 15mm	+/- 25mm	Measured building surveys, high accuracy land & topographic surveys, determined boundaries surveys, Relative Kinematic, PPP-RTK, rapid static, Static Network RTK, PPP, UAV/Drone surveys	1:50	20mm
E	+/- 25mm	+/- 50mm	Measured building surveys, Boundary identification, measured land surveys, topographic surveys, Relative Kinematic, PPP-RTK, rapid static, PPP, UAV/Drone Surveys, LiDaR, Helicopter	1:100	50mm
F	+/- 50mm	+/- 50mm	Boundary identification, measured land surveys, UAV surveys, PPP-RTK, PPP, UAV/Drone surveys, LiDaR, helicopter, fixed wing aerial	1:200	100mm
G	+/- 100mm	+/- 100mm	Topographic surveys, boundary surveys, DGNSS, PPP, UAV/Drone surveys, LiDaR, helicopter, fixed wing aerial	1:500	250mm
H	+/- 250mm	+/- 250mm	Low accuracy topographic surveys, national urban area mapping, DGNSS, UAV/Drone surveys, LiDaR, helicopter, fixed wing aerial, EO	1:1000	500mm
I	+/- 500mm	+/- 500mm	Low accuracy topographic mapping, national non-urban mapping, general boundary mapping, DGNSS, fixed wing aerial, EO	1:2500	1000mm

Table 6: GNSS survey accuracy table (based on the original Measured surveys of land, buildings and utilities, 3rd edition). *To be used in conjunction with tables 1 & 2. Accuracies are given as two sigma (σ)*