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GNSS Surveying for GIS

- Basudeb Bhatta

Sr. Systems Engineer, Computer Aided Design Centre, Jadavpur University, Kolkata – 700 032, email: basu@bbhatta.info

1 Introduction

Geographic Information System (GIS) is defined as an information system that is used to input, store, retrieve, manipulate, analyze and output geographically referenced data or geospatial data, in order to support decision making for planning and management of land use, natural resources, environment, transportation, urban facilities, and other administrative records. GIS makes map creation and information retrieval efficient and easy. GIS involves layering the data about a particular site. In a GIS study, the land survey is the first layer, creating the framework for additional layers that give more detail, such as the exact positions of street signs, traffic lights, telephone poles, and fire hydrants in a city.

The power of a GIS comes from the ability to relate different information in a geospatial context and to reach a conclusion about this relationship. Most of the information we have about our world contains a location reference, placing that information at some point on the globe. A GIS that can use information from many different sources in many different forms, can help with various analyses in reference to geographic locations. The primary requirement for the source data consists of knowing the locations for the variables; and 'knowing the location' involves surveying. Surveying, as a major source of data in GIS, can involve either the so called 'traditional' methods or the *Global Navigation Satellite System* (GNSS) method or both. More sophisticated information-gathering tools are also being used in GIS. One example is mobile-mapping system. A series of cameras are placed on a vehicle and used in conjunction with a GNSS system. Someone drives the vehicle around the area needing to be mapped. The cameras take video of the road. Later, using the footage, someone can map signs, telephone poles, traffic lights, and other items of interest.

2 Traditional Surveying

Traditional surveying methods are well established and being reliably used since a very long time. Prior to the advent of GNSS surveying, these surveys provided the basis for information. There are three types of traditional surveys commonly used: triangulation, traverse, and leveling.

Triangulation Surveys A triangulation survey determines the position of a point by measuring vertical and horizontal angles to control points using an instrument called a theodolite. Because this method requires clear lines-of-sight, control points are generally placed at the greatest available elevations. In flat or obstructed areas, large towers are sometimes constructed to help assured line-of-sight. Also, distances are measured at intervals to provide scale to the survey. It often takes days to acquire the observations and longer to process them.

Traverse Surveys A traverse survey determines the position of a point relative to an out-of-sight control point by creating a cumulative series of distance and angular measurements from the control point to the point to be positioned. Originally, distances were measured with chains, bars, or tapes. Nowadays, electronic distance meters (EDM) are used, which uses microwave, light, and laser ranging to determine distances. These instruments provide positions good to about 10 cm to a normal range of about 50 km. A clear line-of-sight is still required in traverse surveying, meaning that a high number of measurements are needed to complete a survey. Thus, the cost and complexity of the survey is often huge, and errors accumulate as more distances and angle measurements are necessary to connect to the new control point.

Leveling Surveys A leveling survey determines the heights of locations above or below a model of the earth known as *geoid*. There are three different types of leveling surveys: *differential*, *trigonometric*, and *barometric*. The most accurate method of leveling is *differential leveling*. In this method, the height of one location is measured relative to the known height of another location. Readings are made on graduated rods held in an upright position ahead of and behind a carefully leveled instrument. The difference between the readings is the difference in elevation between the points. *Trigonometric leveling* involves measuring a vertical angle from a known distance with a theodolite and computing the elevation for the point using trigonometry. This method permits the surveyor to do both triangulation and leveling at the same time, which is efficient, but less accurate, than differential leveling. *Barometric leveling* involves measuring the differences in atmospheric pressure between points in the survey. While barometric leveling is the least accurate of the leveling techniques, it can rapidly provide relative heights. Further, differential and trigonometric leveling require clear lines-of-sight, thus increasing the number of measurements needed and the cost and complexity of the survey.

3 GNSS Surveying

GNSS is a satellite based navigation and positioning system. This system provides autonomous spatial positioning with global coverage. A GNSS allows small electronic receiver to determine its location using signals transmitted from navigation satellites. For anyone with a GNSS receiver, the system can provide location (and time) information in all weather conditions, day and night, anywhere in the world.

There are currently two GNSSs in operation: the United States (US) *NAVigation Satellite Timing And Ranging Global Positioning System* (NAVSTAR GPS, commonly referred to as simply GPS) and the Russian *GLObal'naya NAvigatsionnaya Sputnikovaya Sistema* (GLONASS). A third system, *Galileo*, is currently being developed in Europe; and a fourth, *Compass Navigation Satellite System* (commonly referred as *Compass*) has been initiated by China. Other than these global systems there are some regional or local systems as well. GNSS has revolutionized surveying, providing latitude, longitude, and height information more quickly, inexpensively, and accurately than was possible by traditional surveying methods.

GNSS offers an alternative to traditional surveying and can eliminate several of the limitations imposed by traditional surveys. With the help of GNSS, it is now possible to perform surveys much more easily over long ranges and in areas where clear lines-of-sight between points are not available. GNSS observing sessions of only a few hours can yield three-dimensional positions with accuracies of a few centimeters. GNSS antennae can be set up on tripods and connected to separate receivers, or antennae and receivers can even be carried in a backpack arrangement. The surveying method can be static or kinematic, based on the requirement of accuracy. However, the use of GNSS has several limitations. In the real world there are several factors that influence and could make the GNSS performance less than mathematically perfect. These several aspects of GNSS performance are:

- *Accuracy*, at a certain level when the appropriate hardware, software and operational procedures are used.
- *Availability*, the extent to which the system is available to all users, anywhere on the earth, and at any time of the day.
- *Continuity*, the degree to which a certain level of accuracy is maintained on a continuous basis.
- *Reliability* of the system and results, often evidenced by a certain 'repeatability' of the positioning accuracy.
- *Integrity*, the capacity to monitor performance and warn users when accuracy falls below a certain level.
- *Cost*, of hardware and software as well as indirect operational costs.

- *Competitive technologies*, do they exist? What do they offer in terms of superior accuracy, etc.?

However, the most important performance measure for most users is accuracy. Although there are several ways to represent error sources, they can be grouped into three categories, depending on where they take place — (1) errors due to satellite-based uncertainties, (2) errors due to signal propagation, and (3) errors due to receiver-based uncertainties. In addition to these physical errors, a voluntary noise can be introduced, in some constellations, by the system management. Other than different error sources that makes the measurements imperfect, there are some issues involved in the accuracy of GNSS positioning. These are, in strict sense, not considered as errors, but influencing factors to the accuracy matter; for example, number of visible satellites and their arrangements. In the environments where the view of the sky is limited, such as urban areas, the traditional survey techniques must still be used. This is because while GNSS surveying does not require a line-of-sight between points within a survey, it does require unobstructed lines-of-sight to at least four satellites. Further, there are several methods of GNSS surveying (different combinations of static, rapid static, kinematic, standalone, differential, code-based, carrier-based, and so on); and different levels of accuracy can be achieved from these different methods (refer Bhatta (2010) for detailed discussion). Choosing the most effective one often becomes difficult.

4 General Factors for GNSS surveying

The precision of GNSS not only depends on the surveying methods; some general factors are also responsible for the degradation of measurement accuracy irrespective of surveying method(s) adopted. As with most surveying tasks, GNSS surveys are more likely to be successful if properly planned and designed. There are a number of issues that need to be considered before and during performing a GNSS survey. This section presents a number of issues that should be considered before surveys are attempted and during the survey as well. The guidelines in this section are based upon several sources of literature (Jones 1984; Wells 1986; Hoffmann-Wellenhof et al. 1994; Kaplan 1996; Trimble 2001; RMITU 2006; Sickle 2008; Ghilani and Wolf 2008; Bhatta 2008) as well as practical experience. However, it is important to realize that GNSS surveying is an evolving technology. As GNSS hardware, data collection techniques, and processing software are improved, new guidelines will change the existing guidelines.

4.1 Accuracy

Perhaps the first question that needs to be answered pertains to whether GNSS techniques are capable of achieving the accuracy required by the project. Manufacturer specifications indicate that carrier phase surveys are capable of achieving centimetre to sub centimetre accuracy, plus one or two parts per million (ppm) of the baseline length. Surveyors must be aware that these specifications usually correspond to the standard deviation of computed baselines. Doubling (and sometimes greater) the specified value often provides a more realistic assessment of the capabilities of receiver. Users should also be aware that errors due to factors such as multipath are not considered in these accuracy values. Another factor to consider is that the vertical component is, generally, not as accurate as the horizontal component. A rough rule of thumb relates the height error to the horizontal error by a factor of 1.5 to 3.0 depending on satellite geometry.

4.2 Obstructions

In order to apply GNSS technology to surveying applications, a clear view of the satellites is required. This precludes the use of GNSS technology in tunnels, under bridges and in built up areas with tall buildings. In most instances, surveys of features in well established areas may not be completely suitable to GNSS technology. In these situations, portions of the survey, such as the control work, can be performed using GNSS technology. The remainder of the survey can be completed using a total station. Another approach to overcome the problems due to obstruction is offsetting (refer Section 4.12). Professional experience will decide whether sites are suitable or not. It is important to note that the points of interest must be free of overhead obstructions, not simply the area.

4.3 Length of Baselines

One extremely important factor in relative/differential method to consider when planning GNSS surveys is the distance between the two receivers, or *baseline* length. The accuracy of GNSS survey degrades as the separation between the receivers increases. This is due to spatial correlation of errors at both sites not being as high as if the receivers are adjacent to each other. This fact is reflected by the ppm component of most accuracy specifications. In addition, the time required to successfully resolve the integer ambiguity generally increases as the baseline length increases. This results in surveys that are less accurate and not as efficient to perform.

For static surveys, the occupation (or observation) time is generally quite long in order to ensure ambiguity resolution, as well as, to average random measurement and multipath effects. As a result, baseline length is not a critical factor. Users must be aware that a limitation exists if single frequency receivers are used as the ionospheric error will cause problems over baselines greater than 10–15 km. Baseline lengths should be kept below this length. For rapid static surveys, dual frequency receivers are used. The aim of such surveys is to resolve the ambiguities as quickly as possible. The most efficient rapid static surveys are performed when baseline lengths are less than five kilometres.

Kinematic surveys (including real-time kinematic (RTK)) are the most sensitive to baseline length as the resolution of the integer ambiguities in an efficient manner in short occupation time is required. Unsuccessful ambiguity resolution results in surveys that do not meet required accuracy levels. Most manufacturers recommend that RTK surveys be performed over baseline lengths of less than ten kilometres. Although longer distances than this can be observed, best results are achieved when the reference and roving receiver are separated by less than five kilometres.

These guidelines may appear to be restrictive if misinterpreted. It must be cleared that the survey may extend beyond these baseline lengths, it is the reference-rover separation that should stay within these limits. If multiple reference sites are used, surveys can be performed successfully over extremely large areas.

4.4 Occupation Time

For a static survey, the occupation time per point surveyed is selected to provide sufficient measurements to enable the integer ambiguities to be resolved. Users must be aware that a change in satellite geometry during the occupation period is required to enable the ambiguities to be solved. This is partly due to the ambiguity being a value which is extremely close to the distance between the satellite and receiver at the beginning of the survey. As the range to the satellites changes, the ranges and ambiguities start to separate. This enables statistical methods to identify the correct number of integer cycles more easily.

Therefore, 100 measurement epochs (*epoch* is the period or instant of each observation) collected at a one second rate are most likely insufficient to resolve the ambiguities, whereas 100 epochs at a fifteen second rate (i.e. 25 minutes) are more likely to be sufficient. This highlights that occupation time, rather than number of measurements, is the key factor in performing static surveys.

The occupation time required for static surveying is a function of a number of elements including the baseline length, number of satellites, satellite geometry, atmospheric conditions and multipath conditions. In general, using modern technology surveying receivers, 20–30 minutes of dual frequency measurements are usually sufficient to resolve the ambiguities over baseline lengths of less than 10 km. An additional 10 minutes may be sufficient to extend the baselines to 10–20 km. Both these estimates presume continuous tracking of at least five satellites. It should be noted that in the presence of obstructions it may be necessary to increase the occupation time in order to achieve clean measurements. Single frequency users are advised to acquire measurements for twice as long, i.e. 40–60 minutes over baseline lengths of less than 10 km. If there are six, seven, or even eight satellites being observed, experienced users who are familiar with the performance of their equipment may wish to observe for shorter periods than this, say ten minutes, particularly if performing rapid static surveys over shorter baselines. For baseline lengths longer than 25–30 km, dual frequency receivers should be used and observation times should not be shorter than one hour to ensure successful ambiguity resolution.

Kinematic surveys utilize a short initialization technique to resolve the integer ambiguities. This initialization procedure may take several forms. Once the survey is initialized, each point of interest only need be occupied for several epochs. In order to acquire sufficient measurements to detect if a bad epoch has been recorded, surveyors are recommended to acquire at least ten epochs while at rover sites. The recording rate for kinematic surveys is usually higher than that of static surveys, thus, ten epochs of measurement may correspond to less than 30 seconds. To be conservative, occupation periods between 30–60 seconds should be used for kinematic occupations; 40 seconds is standard for the recording rate one epoch/second. Shorter occupation times can be used for detail surveys where the position of the points of interest are only required for plotting features on a survey plan.

4.5 Recording Rate

The recording rate represents the rate at which satellite measurements are stored. This rate is often termed the *data rate* or *epoch rate*. For static surveys, there is little advantage in storing measurements at a high rate. Typically, a recording rate of 10 or 15 seconds is used for static occupation periods of twenty minutes or more. For longer sessions which may involve several hours of measurement, rates of 30 seconds or even one minute are suitable. For static surveying, it is important to assess the amount of work to be performed and weigh the amount of data against the available volume of storage space. For example, it may be feasible to perform four observation sessions of 45 minute duration in one day. In this instance, the user should verify that the rate chosen is such that 180 minutes of data can be stored. It should also be noted that the amount of data storage required will depend on the number of satellites observed and the manufacturer's ability to compress the acquired measurements into efficient data structures. For rapid static surveys, similar considerations apply. The primary difference between a rapid static survey and a static survey is the shortened occupation period. In order to provide the processing algorithm with sufficient measurements to perform statistical operations, a higher data rate is generally used for rapid static surveys. For example, a 10 minute occupation should be performed at a data rate of 5 or 10 seconds, rather than 30 or 60 seconds.

The data requirements of a kinematic survey are quite different from a static or rapid static survey. Kinematic surveys are designed to be more efficient than static surveys by employing shorter occupation times. In general, a minimum of ten epochs at each rover site is recommended. If the data rate is set to 60 seconds, the performance of the kinematic survey is no different from a rapid static survey. A recording rate of at least 3 or 5 seconds is, therefore, usually adopted for stop-and-go kinematic surveys. This enables the roving receiver to occupy marks for less than 1 minute, while still providing sufficient epochs to enable gross measurement errors to be identified. By purchasing additional memory chips or data cards, or using a computer with large hard disk for higher sampling rates can be used. In all cases, an increased cost is incurred. The alternative is to transfer the acquired measurements to a computer during the survey. This requires the use of a computer, generally powered by internal batteries or by a cigarette lighter adapter from a vehicle, to be available in the field. If these options are not feasible, a compromise can be reached by recording at a slower rate, say 10 seconds, and occupying rover sites for closer to 2 minutes.

The final survey type that needs to be considered is the continuous kinematic survey and RTK survey. In these surveys, the position of the receiver while it is in motion is of interest. The recording rate needs to be carefully selected to provide points at desired intervals. For example, a recording rate of 3 seconds will provide one computed position in every 50 m if the host platform is travelling at 60 km/h. The selection of the data rate must, therefore, be computed based on a desired point spacing and the estimated speed of the host platform. Users may find that surveys need to operate at rates of one second to be effective for the chosen application. If so, it may be necessary to invest in additional data storage to enable surveys of a practical observation period to be performed.

One vital point that must be remembered is that the reference receiver must record measurements at least the same rate as the rover. The reference receiver may record faster than the rover, as long as the rate is evenly divisible by the rover rate. For example, a rover rate of ten seconds and a reference rate of two or five seconds is satisfactory. A reference rate of three seconds will mean that two of every three rover epochs are ignored. The measurement epochs in receivers are determined by dividing the GNSS time by the recording rate. A remainder of zero indicates that the measurement should be stored. This means that surveyors do not need to synchronize receivers as such, as the receiver clock performs this function automatically.

4.6 Measurement Redundancy

Professional surveyors prefer redundancy into their survey procedures. Control work is performed by traversing, computing and distributing measurement errors. Radiations are often checked using a right angle offset from a traverse line. Even a detail survey has a check of sorts as anomalous terrain variations and large bends may indicate measurement errors. Each of these techniques provides the surveyor with the ability to detect gross measurement errors. Surveying with GNSS is also similar; the only difference is that the observation procedure is less prone to user error as almost everything is done automatically by the receiver. The most likely source of human error is coordinating incorrect marks or naming marks incorrectly, and erroneous entry of antenna height details. The fact to this is that any errors due to the receiver measurement procedure are often difficult to detect.

GNSS surveys can be designed to contain sufficient measurement redundancy to enable gross errors to be detected. Surveyors should be aware of any requirements that have regarding redundant measurements in GNSS surveys. For example, static control surveys may require the occupation of each mark on at least two separate occasions. For kinematic surveys (including RTK), each point may need to be coordinated from two reference stations. It may also be stipulated that the kinematic occupations are to be independent, in other words, the two reference receivers cannot operate at the same time to enable each point to be occupied once. Surveyors can increase the integrity of their results by planning redundant measurements into their survey procedures. The use of loop closures and least squares adjustments can then be used to isolate problematic measurements (refer Section 4.9 and 4.10).

One additional method by which checks can be built into surveys is to occupy as many previously coordinated marks as possible. A minimum number of control points must be occupied and integrated into surveys to enable coordinates to be computed relative to the appropriate coordinate system. Integration of additional coordinates serves two purposes—it assists in identifying erroneous GNSS baselines, as well as, integrating the survey into the control coordinate system. This also verifies the control coordinates.

4.7 Satellite Geometry

Surveying with satellites is the same with regards to the spacing of the satellites as a terrestrial resection is with regards to the control points. Satellites which are well spaced will tend to provide better results than constellations which are poorly spaced. The indicator used to describe the instantaneous satellite geometry is termed the dilution of precision (DOP). A high DOP value indicates poor satellite geometry; a low value indicates strong geometry. The DOP value is calculated from the inverse of the normal matrix in a point positioning least squares adjustment.

The DOP is highly dependent on the number of visible satellites. Although four satellites can determine a point, a minimum of five satellites should be observed simultaneously for reliable result. During the observing session, the geometric dilution of precision (GDOP) should never be greater than 8. The position dilution of precision (PDOP) should not be greater than 5. Satellite signals shall be observed from a minimum of two quadrants that are diagonally opposite each other. The position of the satellites above an observer's horizon can be obtained from *sky plot* and critical consideration in planning a GNSS survey. A free downloadable program from Trimble for predicting DOP for any location and many constellations (GPS, GLONASS, Galileo, COMPASS, as well as WAAS) is available at *www.trimble.com/planningsoftware.shtml*.

Obstructions that are 20 degrees or more above the horizon should be noted on an obstruction diagram. The effect of obstructions should be minimized by proper reconnaissance prior to observations. Satellite data below an elevation mask of 10 degrees shall not be used in baseline measurements.

4.8 Control Requirements

Almost all surveys require the computed coordinates to be related to an existing set of coordinates. Even a straight forward re-establishment survey will also require the survey to be rotated onto the datum used by a previous survey. This may be performed by setting up on a mark occupied in the previous survey and sighting along a direction determined from the previous survey. This provides the bearing datum, or in effect, determines the necessary rotation parameter to apply to determined coordinates. If the total station being used has been calibrated, the distances can be considered correct.

GNSS surveys are a little different from total station surveys. The coordinates generated from measurements are referenced to the WGS84 datum for GPS (PZ 90 for GLONASS, GTRF for Galileo, and Beijing 1954 for Compass) and are presented in terms of Cartesian coordinate differences between the reference and roving receivers. If the desired coordinate system is different from this, a transformation needs to be applied. For surveying applications where the coordinates of the local control points need to be considered when integrating new points, a global or regional set of pre-determined transformation parameters is often inadequate for application to new points as the parameters are not sensitive to errors in the local coordinates. Therefore, surveyors must occupy points with known coordinates in order to integrate new points into local coordinate systems.

The number of control points required depends on the application. If horizontal coordinates are required, then a minimum of two points are required with known easting and northing coordinates in the desired coordinate system. This enables a scale factor, rotation and two translation components to be computed. Note that any error in the local coordinates will be difficult to detect, as there is no redundancy in the transformation parameter estimation process. It is beneficial to observe a third control point in such circumstances. If the height of points is also required, sufficient information must be available to compute the geoid-spheroid separation. If the survey extends for less than ten kilometres, geometric geoid modelling techniques can usually be applied to good effect. Geometric techniques require the survey to be connected to three points with known horizontal and height coordinates. An additional point with known height only is sufficient to check the success of the geoid modelling technique.

4.9 Loop Closures and Baseline Differences

The GNSS surveying techniques are capable of generating centimetre accuracy results if the carrier phase ambiguities are correctly identified and constrained during data processing. The results are generally presented as Cartesian coordinate differences. These coordinate differences, or vectors, represent the three dimensional coordinate difference between the reference and rover receiver. In addition to Cartesian coordinates, the vectors can be presented in terms of east, north and height differences. This is commonly performed using a local projection. Regardless of the manner in which the vectors are presented, closures of connecting baselines can aid in the detection of erroneous measurements. In the same manner in which a traverse misclose is computed, the three dimensional misclose of GNSS vectors can also be determined. GNSS surveys are not performed to generate traverse measurement equivalents; therefore, surveyors use manually selected baselines to form loops of baselines. The closures can be performed using a calculator; however, some GNSS surveying systems provide loop closure utilities with the data processing software. Intelligent use of loop closures can enable erroneous baselines to be identified.

In order for a loop closure to be performed, baselines are required from more than one observation session. If only one session is used, the baselines are correlated and loop closures will tend to always indicate excellent results. This is due to the correlation between the baselines rather than the quality of the baselines. When multiple sessions are observed, a number of strategies for detecting poor quality vectors can be adopted. Consider the example shown in Fig. 1 where several redundant baselines have been observed. One strategy which may be adopted is to check each triangle while trying to isolate any triangle which reveals poor results. If each triangle is closed, it is likely that a bad baseline will affect more than one triangle. This technique enables checking of correlated baselines from the same session. It is also likely, however, that a session which was too short to enable the ambiguities to be correctly resolved will highlight two low quality baselines. Comparison of all triangles will enable such instances to be detected if sufficient baselines are observed.

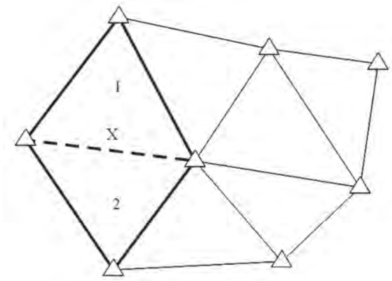

Fig. 1 Redundant baseline observation

In the Fig. 1, if baseline *X* is erroneous, it can be anticipated that triangles 1 and 2 will highlight a poor closure. By performing a closure around the four sided perimeter of triangles 1 and 2, the poor baseline can be highlighted. In addition, several of the points have been occupied on more than one occasion. Performing loop closures will aid in detecting whether antenna height errors are present in the data set.

In order for the processing software to be able to check for poor baselines it is important that the baselines are measured more than once (i.e. over multiple sessions) to obtain independent baselines. For example in the case previously described, the baseline *X* could be measured in the first and second sessions (by holding this baseline fixed), to enable the processing software to do a comparison.

Loop closures and baseline differences in repeat baseline measurements are to be assessed to check for blunders and to obtain initial estimates for internal network consistencies. The software for processing the raw data must be capable of producing results that meet the accuracy standards specified for the survey. The software must be able to produce, from the raw data, relative position coordinates and corresponding variance covariance statistics, which, in turn, can be used as input to three dimensional network adjustment programs.

In case of loop closure analysis, the error of closure is the ratio of the length of the line representing the equivalent of the resultant errors in the GNSS baseline vector components to the length of the perimeter of the figure of the analyzed survey loop. A loop is defined as a series of at least three independent, connecting baselines, which start and end at the same station. Each loop shall have at least one baseline in common with another loop. Each loop shall contain baselines collected from a minimum of two independent sessions. The following minimal criteria for baselines loops shall be used for static and rapid static procedures for first-order classification (1:100000) network adjustment:

- Baselines in the loop shall be from a minimum of two independent observation sessions. Loop closures incorporating only baselines determined for one common observation session are not valid for analyzing the internal consistency of the GNSS network.
- Baselines in the loop shall not total more than 10.
- Loop length shall not exceed 100 kilometres.
- Percentage of base lines not meeting criteria for inclusion in any loop shall be less than 20% of all independent base lines.
- In any component (x, y, z) maximum misclosure shall not exceed 25 cm.
- In any component (x, y, z) maximum misclosure in terms of loop length shall not exceed 12.5 ppm.
- In any component (x, y, z) the average misclosure in terms of loop length shall not exceed 8 ppm.

For repeat baseline difference:

- Baseline lengths shall not exceed 250 kilometres.
- In any component (x, y, z) maximum difference shall not exceed 20 ppm.

4.10 Network Adjustment

When performing networks of GNSS baselines, a least squares adjustment of the generated baselines is often performed once processing is complete. These networks may comprise static and kinematic baselines, however, static baselines are generally observed. The network adjustment procedure has several functions in the GNSS surveying process. The adjustment provides a single set of coordinates based on all the measurements acquired, as well as, providing a mechanism by which baselines that have not been resolved to sufficient accuracy can be detected. A series of loop closures should be performed before the network adjustment procedure to limit the number of erroneous baselines entering the adjustment process. A further feature of the network adjustment stage is that transformation parameters relating the GNSS vectors to a local coordinate system can be estimated as part of the adjustment. The adjustment process can be done in several ways. The following are the major elements of the adjustment process.

4.10.1 Minimally Constrained Adjustment

Once the processed Cartesian vectors have been loaded into the adjustment module, an adjustment should be performed where no coordinates are constrained. The adjustment should be performed using the datum of respective GNSS constellation (e.g., WGS84 for GPS). In actual fact, the processor does constrain one point internally to enable this adjustment to be solved. This solution provides a mechanism by which GNSS baselines which are not sufficiently accurate can be detected. Once the minimally constrained adjustment has been performed, the surveyor should analyze the baseline residuals and statistical outputs (which will differ between adjustment programs) and ascertain whether any baselines should be removed from subsequent adjustments. This process relies on the baseline network being observed in such a manner to ensure that redundant baselines exist. It is the redundant baselines that enable erroneous baselines to be detected.

4.10.2 Constrained Adjustment

Once the minimally constrained adjustment has been performed and all unsatisfactory baseline solutions removed, a constrained adjustment can be performed. The constrained adjustment is performed to compute transformation parameters, if required, and yield coordinates of all unknown points in the desired coordinate system. The surveyor must ensure that sufficient points with known coordinates are occupied as part of the survey. The user should analyze the statistical output of the processor to ascertain the quality of the adjustment. Large residuals at this stage, after the minimally constrained adjustment has been performed, will indicate that the control points are non-homogeneous. It is, therefore, important that additional control points are occupied to ensure that such errors can be detected.

4.10.3 Error Ellipses

The standard deviations of the estimated coordinates are derived from the inverse of the normal matrix generated during formulation of the least squares process. Error ellipses for each point can be computed from the elements of this matrix. The ellipse presents a one standard deviation confidence region in which the most probable solution based on the measurements will fall. Surveyors should base the quality of the adjustment process on the magnitude of these ellipses. Many contracts will specify the magnitude of error ellipses for both the minimally constrained and fully constrained adjustments as a method of prescribing required accuracy levels. The product documentation for the adjustment program will further indicate the manner in which the ellipse values are generated.

4.10.4 Independent Baselines

For the least squares adjustment process to be successful, the surveyor must ensure that independent baselines have been observed. If more than one session is used to build the baseline network, then independent baselines will exist. In instances where one session is observed and all baselines adjusted, the measurement residuals will all be extremely small. This is due to the correlation that exists between the baselines solutions as they are derived from common data sets. This is not a problem as long as the surveyor is aware of the occurrence and does not assume that the baselines are of as high accuracy as implied from the network adjustment results. The inclusion of independent baselines is an important component of GNSS survey design and leads to a strong network configuration.

4.11 Independent Reoccupation of Stations

GNSS surveys require redundancy of observations which are used to detect blunders and to obtain statistically sound results. Redundancy is achieved by reoccupying some points in different sessions with different geometric combinations. The following criteria pertain to static, rapid static and reoccupation procedures for network adjustments:

- 10% of all stations should be occupied three times or more.
- All vertical control stations should be occupied twice or more times.
- 25% of published horizontal control stations should be occupied twice or more times.
- All ‘station pairs’ for azimuth control should be occupied simultaneously twice or more times.
- 100% of new stations should be occupied twice or more times.
- For sessions where stations are occupied in succession, the antenna/tripod must be physically moved and reset between the sessions to be classified as an independent occupation.

4.12 Point or Line Offset

One technique to avoid multipath and signal attenuation is offsetting (Fig. 2). Offsetting can also be adopted where the survey-location is inaccessible due to any reason. The offset point must be established far enough from the original position to avoid an obstructed signal, but close enough to prevent unacceptable positioning error. The *length of tie* (distance between the original point and offset point) may be measured by an external laser, a laser cabled directly into the GNSS receiver, or even a tape and clinometers (for measuring angles).

A technique unique to RTK and differential GNSS, and used especially in mobile GNSS applications, is the creation of dynamic lines. The GNSS receiver typically moves along a route to be mapped logging positions at predetermined intervals of time or distance. These points can then be joined together to create a continuous line. Obstructions along a route present a clear difficulty for this procedure. Where it is impossible or unsafe to travel along the line to be collected in the field the dynamic line may be collected with a consistent offset (Fig. 2b). This technique is especially useful in the collection of roads and railroads where it is possible to estimate the offset with some certainty due to the constant width of the feature. It is also possible to collect routs with individual discrete points with short occupations where that approach recommends itself.

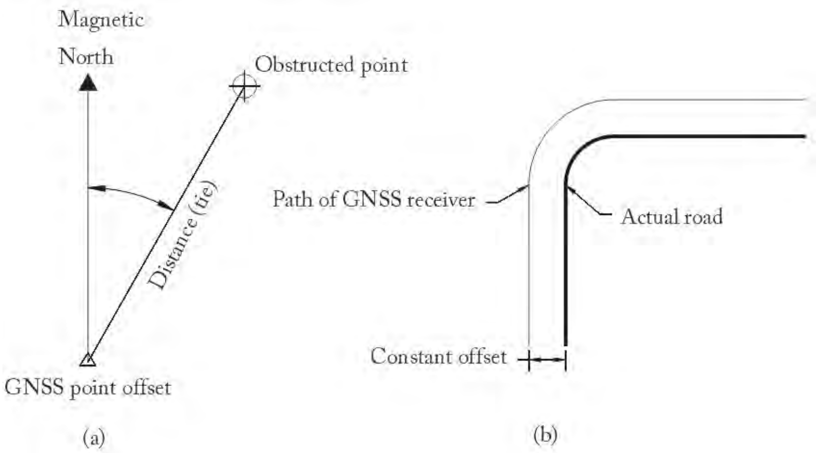

Fig. 2 Point offset (a) and line offset (b)

4.13 Float Solution

If insufficient data has been acquired to successfully resolve the carrier phase ambiguities, a float solution is generated. In the float solution, the ambiguities are not constrained to integers, rather are left to 'float' as real numbers. Most commonly, the floating ambiguities will not be close to integers in this instance. The most precise results are obtained when the ambiguities are constrained to integers; therefore, a float solution generally implies that the required accuracy will not be met. In most cases, the baseline will need to be re-observed, however, there are some processing modifications that may be used to alleviate this problem. This discussion assumes that the survey has been performed using the static or rapid static observation technique. Kinematic techniques are generally much harder to resolve and are best to be re-observed.

The surveyor should closely analyze the output provided by their data processing program. In all float solution cases, the ratio of the sum of the squares of the residuals for the potential solutions will be a low number close to one. This indicates that there is no clear solution to the integer ambiguities. The meaning and nature in which the ratio is computed will depend on the processing package and users should refer to their documentation. If the ambiguities are displayed as numeric values, they will be real numbers which do not approach integers. In addition, the measurement residuals may be large. In the instance that every ambiguity does not appear to be an integer, the baseline is probably best re-observed. If, however, there is one satellite which is not close to an integer, but the other values are quite close, the data should be re-processed after eliminating this satellite from processing. Similarly, analyzing the residual graphs may reveal one or two noisy satellites which can be eliminated from processing to try to generate a fixed ambiguity solution.

Another modification worth trying is raising the elevation mask. If data has been observed at ten degrees and residual graphs reveal that measurements are particularly noisy when satellites are low to the horizon, the mask can be raised to, say, fifteen degrees and the processor run again. This may alleviate the problem and generate fixed ambiguities.

In general, when problems during processing occur that cannot be resolved using the above suggestions, the baseline must be re-observed. However, by analyzing the output provided from the processing program and looking for satellite measurements which may be causing problems, some baselines may be able to be processed to a considerable accuracy.

5 Conclusions

The evolution of surveying from chains, bars, tapes, theodolites, and levels through the EDM to the GNSS antennae and receivers of today has produced a dramatic increase in the speed and accuracy with which positioning can be accomplished. In most of the cases, digital data from a GNSS receiver can directly be fed into the GIS system and be analyzed. The cost and complexity of surveying has also decreased with the use of GNSS surveying techniques. However, the GNSS system also has several limitations. Scientists and technologists are continuously working to overcome these limitations. It is hoped that GNSS, in near future, will be used in every survey operation with increased reliability and accuracy.

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