PROCEDURE FOR GNSS EQUIPMENT VERIFICATION IN STATIC POSITIONING

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***Abstract**: GNSS static positioning is the premier method for numerous surveying applications requiring high precision. Many countries have developed national guidelines to specify the minimum requirements for the best practise and statistical evaluation for static GNSS surveying, but there is still missing a uniform standard that can be adopted by the surveyors to assess their equipment prior to any surveying task. This paper aims to describe a guideline that promotes uniform procedures for equipment verification and data assessment in order to achieve the highest level of integrity for routine surveying applications. The guideline refers to field practise and statistical evaluation of the measured data using the local and the global tests. The former is an evaluation procedure performed on individual survey measurements, e.g. baseline, in order to assess the quality of a measurement and its assumed uncertainty at a statistical level. The latter is an evaluation procedure performed to assess the quality of the survey, such as a network, as a whole. Results from a number of field tests are given using different types of geodetic receivers*

***Keywords:** GNSS surveying, static positioning, verification, receivers*

1. Introduction

In the three decades that Global Navigation Satellite Systems (GNSS) have been employed for precise positioning work, the technology has profoundly influenced the practice of land surveying. It is interesting to note that GNSS innovation for surveying and mapping has been less about improved accuracy or precision and more about faster and more easily validated positioning, and decreasing costs. This trend has resulted in the use of real-time kinematic (RTK) positioning and the development of networks for continuous operation of GNSS receivers. It seems that the accuracy needs within the surveying and mapping community for many applications have been met, while more and faster data collection is yet an unfulfilled demand.

Regardless of the maturity of GNSS it is still essential to ensure the precision of the measurements. Whilst a surveyor will always endeavour to obtain the most accurate and precise measurements to survey control marks, the true value of a survey control mark's position can never be measured nor derived with absolute certainty due to the inescapable presence of measurement error. In order to determine the single, most reliable position from a range of measurements whilst at the same time detecting and removing unacceptable measurement errors, it is necessary to employ instruments that are calibrated and assessed for their stated accuracy.

Having recognised the need for specifications regarding the best practices for GNSS positioning many countries (e.g. USA, UK, Australia etc) have developed series of publications for the surveying community to promulgate specifications for geodetic control surveying using the emerging capability of positioning with GPS in a static or kinematic mode. All the above efforts express valuable guidelines for the practical and experimental use of geodetic-type receivers but they lack of standardized test procedures. In addition, these procedures are not always easily tractable and the time and effort required to perform these are not insignificant.

One example of standardised procedures is the series of ISO (International Organization for Standardization) standards “Optics and optical instruments—field procedures for testing geodetic and surveying instruments” [ISO-17123-(1-8), 2012]. In fact the ISO 17123 part 8 refers to RTK field and data analysis procedures to check the equipment but there is no ISO guideline for using receivers in the static mode. Clearly, there is a need for an international organisation like ISO to expand their standards and provide an outcome based framework that supports the highest level of rigour and integrity in the delivery of static positioning results. The aim of this paper is to present some guidelines based on the Australian specifications (ICSM, 2014a) that can be implemented by the surveyors in order to verify their GNSS equipment prior to any positioning task involving survey control networks. The paper is organised is four sections. In particular, in section 2 a brief description of the guidelines is given, and in section 3 the field tests and data analysis is described. Section 4 provides the test results and their statistical evaluation and concluding remarks are drawn in section 5.

2. Evaluation of a GNSS control survey

GNSS based surveys can vary in quality, depending on the type of GNSS receiver, antenna, ancillary equipment and observation parameters chosen. All have a direct influence on the uncertainty of derived GNSS measurements. Similarly, variation in measurement uncertainty will result from the spatial extent of the survey, the presence of certain external influences and the environmental conditions under which the control survey is undertaken. For this reason, the formulation of accuracy classifications for GNSS control surveys like the ones used for conventional surveying techniques has been shown to be less practical. Instead, the quality of a control survey is quantified in terms of survey uncertainty (SU), positional uncertainty (PU) and relative uncertainty (RU).

SU is the uncertainty of the horizontal and/or vertical coordinates of a survey control mark relative to the survey in which it was observed and is free from the influence of any imprecision or inaccuracy in the underlying datum realisation. Therefore, SU reflects only the uncertainty resulting from survey measurements, measurement precisions, network geometry and the choice of constraint. A minimally constrained least squares adjustment is the preferred and most rigorous way to estimate and test SU at the 95% confidence level. PU is the uncertainty of the horizontal and/or vertical coordinates of a survey control mark with respect to the defined datum and represents the combined uncertainty of the existing datum realisation and the new control survey. That is, PU includes SU as well as the uncertainty of the existing survey control marks to which a new control survey is connected. A fully constrained least squares adjustment is the preferred and most rigorous way to estimate and test PU at the 95% confidence level. RU is the uncertainty between the horizontal and/or vertical coordinates of any two survey control marks. Such marks may be connected by measurement directly or indirectly. The preferred and most rigorous means for deriving RU between pairs of marks is by propagating the respective variances and co-variances obtained from a minimally or fully constrained least squares adjustment (i.e. from SU or PU).

The SU is adopted as the most intuitive and useful metric for classification of geodetic control accuracy and the Australian guidelines recommend two categories depending on the baseline lengths and the duration of observation sessions. Thus, the first category refers to SU being less than 15mm for horizontal position and less than 20mm for ellipsoidal height and the second category refers to SU being less than 30mm for horizontal position and less than 50mm for ellipsoidal height (ICSM, 2014b).

In order to evaluate the quality of a survey and the derived results two tests are used; the local test and the global test. A local test is an evaluation procedure performed on individual survey measurements to assess the quality of a measurement and its assumed (or *a-priori*) uncertainty. To validate each measurement and assumed uncertainty, the size of each adjusted measurement correction should be tested to verify that the correction lies within the upper and lower limits of the specified confidence interval. The local test is conducted using the normal distribution at the 95% confidence level. A correction which exceeds the upper 95% confidence limit indicates a failure and the need to re-evaluate the assumed uncertainty of the measurements and/or imposed constraints. The local test is intended to quantify the repeatability that a surveyor should expect when measuring between two adjacent points. The global test is an evaluation procedure performed on a least squares adjustment to assess the quality of the survey as a whole.

3. Field tests

3.1 Procedure

The recommended procedure for the verification of GNSS equipment comprises three tests: a) a zero baseline test, b) an EDM baseline comparison test, and c) a network test (ACT Government, 2011).

A zero baseline test can be used to determine the precision of the receiver measurements, cabling and the data processing software, and hence the correct operation of the surveyor's GNSS system. The test should be performed for all pairs of receivers when the GNSS equipment is first acquired, immediately after any repairs, and before commencing high precision surveys. This test is performed using a special antenna-cable splitter so that the majority of biases cancel during processing and the quality of the resulting baseline is only a function of random observation error (or noise), and the propagation of any individual receiver biases.

The EDM baseline test allows to make a comparison of the GNSS derived distances against the certified distances. The baseline measurements are repeated with varying satellite geometry. Pairs of observed values are reduced to spheroidal chord distances and compared against the certified distances. Acceptance criteria for the distances are 5 millimetres or better at the 95% confidence interval.

The network test comprises four stations and each station is connected to the others with a minimum of three independent baselines. The local and the global tests are implemented to verify the acceptance criteria at a specified statistical level.

3.2 Data collection

The field tests comprised GNSS static observations acquired at permanent monumentation located at the university campus. Specifically, the stable and concrete pillars are situated on the top of the roofs of selected buildings in the campus. Four pillars were chosen for the experiments which are: LAMG, EST, FYS, GEN (Fig. 1). The relative distances between the points are given in Table 1.

Fig.1. View of the four control points

The equipment used for the data collection is a set of two geodetic receivers Trimble 5800 with quoted accuracy of 5mm ±0.5ppm horizontally and 5mm ± 1ppm vertically and a set of two geodetic receivers Leica 1200+GNSS with quoted accuracy of 5mm ±5ppm horizontally and 10mm ± 5ppm vertically. The zero-baseline test was not performed due to the inability of the specific receivers to connect to the splitter cable.

The baseline test involved the observation of the baseline LAMG-EST for duration of 40min at four different sessions. Each observing session had at least 90min difference from each other. Observing GNSS at different sidereal times, with different satellite configurations, and different atmospheric conditions, is the strongest defence against systematic errors and excessive random errors. Two independent observations in reasonable agreement have a high likelihood of minimal systematic and random error, with increased ability to isolate blunders. The network involved the measurements of all possible combinations of baselines for duration of 20min. Based on best practise procedures that are widely recognized as capable of achieving stated levels of accuracy and known to produce high quality work, but without being a mandate, it was decided to use as observation interval for all the tests equal to 10sec, elevation mask set to record down to zero degrees elevation, and the antenna orientated to within 5 degrees of true North.

Table 1 provides information on the baseline lengths, and the number of visible GPS and GLONASS satellites during the tests, for both days. The tests were performed in two days, at the same time for maintaining the satellite geometry, to accommodate the two different types of receivers.

Table 1. Information regarding the data collection during the field tests

Baseline		**Baseline distance (m)**	**Observation session (min)**	**Number of visible satellites**			
Base	**Rover**			**Base**		**Rover**	
				GPS	GLONASS	GPS	GLONASS
Lamg	Est	150	40	20	16	17	16
	Fys	300	20	13	9	11	8
	Gen	310	20	13	10	11	8
Est	Lamg	150	40	17	15	20	16
	Fys	370	20	11	10	11	7
	Gen	150	20	13	10	11	6
Fys	Lamg	300	20	13	9	10	8
	Est	370	20	13	9	11	6
	Gen	90	20	13	9	11	5
Gen	Lamg	300	20	13	8	10	8
	Est	150	20	13	8	11	6
	Fys	90	20	13	8	11	6

4. Data processing and analysis

The data processing was performed using the GNSS post-processing software utility Grafnav and Grafnet which is more powerful than common commercial software. All the results, produced using double-differenced carrier phase observations and navigation type ephemeredes, were referred in ITRF08 and subsequently in WGS84 and this is the reference system in the results shown in all the following tables. The steps discussed below refer to (a) independent baseline processing, (b) local test with a minimally constrained laser squares solution (c) global test with a minimally constrained and a fully constrained least squares solution.

4.1 Baseline processing

Initially, the test procedure was performed whereby individual baseline measurements are assessed for their quality and their assumed (or *a-priori*) uncertainty. To validate each measurement and assumed uncertainty, the measured baseline is compared with known coordinates and the size of each measurement correction is checked. This testing demonstrates that no gross errors exist within the measurements and all measurements satisfy any predefined measurement precision criteria.

The baseline “Lamg-Est” was observed 40min from each station four times and with both sets of receivers. Table 2 shows the results of the differences in the baseline components from the known coordinates.

Table 2. Baseline repeatability

Differences in baseline components (m)							
		Trimble 5800			Leica 1200		
Base	**Rover**	**ΔX_i-ΔX^{known}**	**ΔY_i-$\Delta Y_k{}^{nown}$**	**ΔZ_i-ΔZ^{known}**	**ΔX_i-ΔX^{known}**	**ΔY_i-ΔY^{known}**	**ΔZ_i-ΔZ^{known}**
Lamg	Est	0.001	-0.001	0.002	-0.004	-0.002	-0.007
Est	Lamg	0.000	-0.002	-0.002	0.004	-0.003	0.006
Est	Lamg	-0.001	-0.001	0.001	0.003	0.003	0.008
Lamg	Est	-0.001	0.002	-0.003	-0.007	-0.002	-0.004

The repeatability in the baseline tests is 0.001m, 0.002m and 0.002m in ΔX, ΔY, ΔZ respectively, for the Trimble measurements, and 0.005m, 0.002m and 0.007m in ΔX, ΔY, ΔZ respectively, for the Leica measurements. The Trimble results are slightly better due to the higher number of observed satellites. For the given baseline distance, the accuracy is given as 1.5×10^{-5} ppm for the Trimble measurements and.5×10^{-5} ppm for the Leica measurements which are well within the manufacturers’ specifications.

The second part of the analysis refers to the network data which are processed initially on a baseline basis and then a least squares adjustment is performed to assess the quality of the survey data as a whole. This is because the completed network must provide the following:

- Elimination or reduction of known and potential systematic error sources.
- Sufficient redundancy and testing to clearly demonstrate the stated accuracy.
- Adequate data processing and analysis.
  Sufficient documentation to allow verification of the results

The network refers to the stations “Lamg- Gen- Est-Fys”. The processing of the baselines from each set of receivers was performed separately and the results were compared again to the known coordinates. Prior to any comparison and further processing, loop closures checks were also performed to eliminate blunders. Table 3 provides the closure errors in X, Y, Z and the total closure error, for the two types of receivers. The total closure errors did not exceed the 0.009m in both loops.

Table 3. Loop closure errors

Loop closure errors				
	Lamg-Fys-Gen-Lamg		**Lamg-Est-Gen-Lamg**	
	Trimble 5800	Leica 1200	Trimble 5800	Leica 1200
CX (m)	-0.004	-0.002	-0.003	-0.004
CY (m)	-0.004	-0.001	-0.001	-0.005
CZ (m)	0.005	-0.003	-0.002	-0.007
CE (m)	0.0075	-0.004	0.0037	0.009

Table 4 provides for the three baselines “Lamg- Gen”, “Est-Fys”, “Fys-Gen” the relative differences between the baselines components obtained from the two types of receivers.

Table 4. Relative differences in baseline components (Trimble 5800 – Leica 1200**)**

Baseline Differences in ΔX, ΔY, ΔZ				
Base	**Rover**	**ΔX^{Tr}-ΔX^{L} (m)**	**ΔX^{Tr}-ΔX^{L} (m)**	**ΔX^{Tr}-ΔX^{L} (m)**
Est	Fys	-0.003	0.000	0.007
Fys	Est	0.006	0.010	0.004
Fys	Gen	0.015	0.002	0.009
Gen	Fys	-0.013	-0.001	-0.011
Gen	Lamg	-0.012	-0.004	-0.010
Lamg	Gen	0.019	-0.018	-0.019

The rms of the differences is 0.014m, 0.009m and 0.012m for ΔX, ΔY, ΔZ respectively. The ppm accuracy for the given distances do not exceed the 2 and the 2×10^{-5} ppm for the Trimble measurements and.4×10^{-5} ppm for the Leica measurements which are well within the manufacturers specifications.

4.2 Network processing

Following the independent baseline processing, the network processing was performed using minimally constrained adjustment in order to apply the local test. The minimally constrained solution involved 42 measurements and the a posteriori variance factor was estimated as 1.002 for both receivers. Using the local test, each baseline is examined to assess whether the correction exceeds the 95% critical value. Specifically, the procedure examines whether the normalised residual exceeds the critical value of the unit Normal distribution at 95%, which is 1.96.

In table 5, an example of the local test baseline assessment is given, only for the Leica data. Specifically, the corrections in ΔX and ΔY are calculated by comparing the baseline component with the adjusted by the least squares process component, and then the normalised residuals are calculated by dividing the measurement correction by the standard deviation of the correction. It is seen that the baseline "Gen-Lamg" exceeds the critical value of 1.96 and therefore is rejected.

After the minimally constrained solution with station "Lamg" being considered as fixed, the a posteriori sigma zero was computed as 0.999 (for Trimble) and 1.000 (with Leica) which are acceptable results. When the a posteriori sigma zero is not close to unity, as a first attempt to rectify such failures is to rescale all GNSS measurement uncertainties. This is based on the assumption that all GNSS baselines were derived under the same conditions and there are no gross errors. Following scaling of all measurements, the minimally constrained adjustment produces a new sigma zero value of unity. In this case though, there was no need to rescale the baselines because the sigma zero value is not significantly less than unity. Therefore, there is no indication that the system of measurements is more or less precise than indicated by the supplied uncertainties and it was decided to exclude the specific baseline "Gen-Lamg" from the subsequent process in the global test.

Table 5. Baselines components for the local test (in m

Baseline		**Correction ΔX**	**Correction ΔY**	**Correction St. deviation**	**Norm. Residual X**	**Norm. Residual Y**
Lamg	Est	0.000	0.000	0.0020	0.000	0.000
Lamg	Est	-0.001	-0.001	0.0020	0.500	0.500
Est	Lamg	0.000	0.000	0.0020	0.000	0.000
Est	Lamg	-0.001	-0.001	0.0020	0.500	0.500
Est	Fys	-0.001	-0.001	0.0025	0.392	0.392
Est	Gen	0.001	0.001	0.0032	0.316	0.316
Lamg	Fys	0.000	0.000	0.0025	0.000	0.000
Lamg	Gen	0.000	0.000	0.0025	0.000	0.000
Gen	Fys	0.001	0.001	0.0030	0.333	0.333
Fys	Lamg	-0.005	-0.005	0.0030	1.667	1.667
Fys	Est	-0.004	-0.004	0.0030	1.333	1.333
Fys	Gen	0.000	0.000	0.0025	0.000	0.000
Gen	Est	-0.003	-0.003	0.0030	1.000	1.000
Gen	Lamg	-0.006	-0.006	0.0030	2.000	2.000

Table 6 provides the computed survey uncertainty of the network stations using for both types of receivers. The results (in m) of the table indicate that the specific survey is well within the specifications, i.e. SU should be less than 15mm for horizontal position and less than 20mm for ellipsoidal height (as discussed in section 2).

Table 6. The computed Survey Uncertainty error (m) at 95% level (minimally constrained**)**

	Trimble 5800			Leica 1200		
Stations	**SU (E)**	**SU (N)**	**SU (Up)**	**SU (E)**	**SU (N)**	**SU (Up)**
Lamg	constrained			constrained		
Est	0.0123	0.0120	0.0126	0.0087	0.0085	0.0090
Fys	0.0118	0.0122	0.0129	0.0091	0.0088	0.0089
Gen	0.0122	0.0119	0.0125	0.0085	0.0086	0.0087

Having confirmed all measurements pass the local test, constraints are imposed on the adjustment to propagate datum and uncertainty from the national grid system. The national grid refers to the Greek Reference System of 1987. The fully constrained solution involved two fixed stations, “Lamg” and “Est”. The a posteriori sigma zero was 0.828 for Trimble measurements and 0.999 for the Leica measurements and the variance factor was estimated as 1.459 for the Trimble measurements and 1.002 for the Leica measurements.

Table 7 provides for the two stations “Gen” and “Fys”, the differences in the coordinates X, Y, Z between the initial and the adjusted values for both types of receivers.

Table 7. Differences (in m) in coordinates for both types of receivers

Station	X^{known}-X^{adj} (Trimble)	X^{known}-X^{adj} (Leica)	Y^{known}-Y^{adj} (Trimble)	Y^{known}-Y^{adj} (Leica)	Z^{known}-Z^{adj} (Trimble)	Z^{known}-Z^{adj} (Leica)
Fys	-0.003	-0.008	-0.002	-0.002	-0.002	-0.002
Gen	-0.002	0.005	0.001	0.004	-0.004	0.005

Table 8. The computed Survey Uncertainty error (m) at 95% level (fully constrained**)**

	Trimble 5800			Leica 1200		
Stations	**SU (E)**	**SU (N)**	**SU (Up)**	**SU (E)**	**SU (N)**	**SU (Up)**
Lamg	constrained			constrained		
Est	constrained			constrained		
Fys	0.0119	0.0123	0.0128	0.0090	0.0086	0.0088
Gen	0.0122	0.0120	0.0125	0.0086	0.0087	0.0087

Again, the results (in m) of Table 8 indicate that the specific survey is well within the specifications i.e. SU should be less than 15mm for horizontal position and less than 20mm for ellipsoidal height (as discussed in section 2). In addition to the results of table 8, the circular radius has also been computed for all baselines in the fully constrained solution and there was no value greater than 6mm for both types of receivers.

This adjustment yields a pass in the global test and a pass in the local test for all measurements and constraints. As inferred from the sigma zero values, the system of measurements were better than indicated by the prescribed uncertainties, and none of the constraints was shown to bias the adjustment in a significant way or to cause any of the measurements to fail.

5. Concluding remarks

Approved methodologies for establishing legal traceability of position determined by GNSS receivers currently do not exist under any official international organisation. Therefore, many countries, such as Australia recommend that GNSS derived positions and distances should not be used as the sole method of measurement during a survey. For surveys where GNSS equipment is used, surveyors are strongly encouraged to adopt best practice.

Guidelines for best practise have been prepared by various countries to assist surveyors who use GNSS equipment and require verification of its performance. However, it is important to have a guideline that promotes uniform procedures for equipment verification and data assessment in order to achieve the highest level of integrity for routine surveying applications. Based on the Australian guidelines, this paper has followed a consistent procedure for testing GNSS equipment so the results obtained provide a reliable verification.

The use of two types of geodetic receivers showed that both performed well within the manufacturers' specifications. The baseline test showed that the repeatability is high in both types of receivers (from 1-7mm) and the rms of the differences from the true values do not exceed the 1cm. The network test verified that the specific survey results in both the local and global tests are well within the specifications at the 95% confidence level i.e. SU should be less than 15mm for horizontal position and less than 20mm for ellipsoidal height.

With the view of standardising the procedures as described and implemented in this paper, it is critical to update the series of ISO 17123 part 8 ISO standards that refers to RTK field and data analysis procedure in order to include the GNSS equipment verification in static positioning.

6. References

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