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Precision and accuracy of the static GNSS method for surveying networks used in Civil Engineering

Precisión y exactitud del método GNSS en modo estático para redes topográficas utilizadas en ingeniería civil

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ABSTRACT

A field check was implemented for calibrating surveying equipment. It was geo-referenced with a Total Station Theodolite and by implementing procedures concerning repeatability and reproducibility. We carried out GNSS (Global Navigation Satellite System) static positioning with double frequency equipment, sensitizing occupation times, day times, uncorrected coordinates subjected to a differential correction procedure and type of coordinates obtained. This facilitated an evaluation of precision and accuracy for the GNSS positioning with the static method, which gave a global RMSE (root mean square error) of 1 cm for conditions with no multi-path effect and 4 cm for field calibration points close to buildings. Additionally, optimal results for occupation times of 30 minutes were found, and the need to use planar Cartesian coordinates to ensure compatibility with the surveys using electronic measurement of distances, which allows the use of the static GNSS positioning for geo-referencing precise surveying networks, and can be used in different applications in Civil Engineering.

Keywords: GNSS, Static GNSS, precision, accuracy, multi-path effect, surveying networks.

RESUMEN

Se implementó un campo patrón de verificación de equipos topográficos. Este fue geo-referenciado con un instrumento de estación total y mediante procedimientos de repetibilidad y reproducibilidad. Se realizaron posicionamientos estáticos GNSS (por las siglas en inglés de *Global Navigation Satellite System*) con equipo doble-frecuencia, sensibilizando el tiempo de ocupación, la hora del día, las coordenadas sin corregir y sometidas al procedimiento de corrección diferencial y el tipo de coordenadas obtenidos. Lo anterior permitió una evaluación de la precisión y exactitud del posicionamiento GNSS con el método estático, encontrándose un error medio cuadrático global de 1 cm para condiciones sin efecto de multi-trayectoria y de 4 cm para los puntos del campo de verificación cercanos a edificios. Adicionalmente se encontraron resultados óptimos para tiempos de ocupación de 30 minutos y la necesidad de utilizar coordenadas planas cartesianas para garantizar la compatibilidad con los levantamientos utilizando la medición electrónica de distancias, lo cual permite utilizar el posicionamiento GNSS estático para georreferenciar redes topográficas de precisión y pueden ser usadas para diferentes aplicaciones en ingeniería civil.

Palabras clave: GNSS, GNSS estático, precisión, exactitud, efecto multi-trayectoria, redes topográficas.

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Introduction

GNSS (Global Navigation Satellite System) has become an important technology because it certifies the presence of positions, from collected and designed items, in a global reference system, thus ensuring inter-operable infrastructure projects (Bernabé *et al*., 2012).

GNSS surveying with the static method is widely used to calculate high-precision tridimensional coordinates in traverse stations: these systems provide coordinates of ground locations at a millimeter level both in the horizontal and vertical components. In addition, the static GNSS positioning allows to accurately determinate the azimuth, for establishing the network's orientation with respect to the reference system. A major advantage of installing surveying networks with GNSS positioning is that it does not require inter-visibility, as compared to others built with electromagnetic distance measuring devices (Jackson *et al*., 2011)its gravity field, and geodynamic phenomena (polar motion, Earth tides, and crustal motion.

The main applications of GNSS networks in static mode are setting control points for monitoring deformation of structures (Rizos *et al*., 2003), as well as constructing base traverses for linear objects such as roads, railways and

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flow media lines (Zhang *et al*., 2014)such as highways and railways, by applying a vehicle-borne GPS/INS kinematic surveying system which integrates the Global Positioning System (GPS.

However, besides the known advantages of satellite navigation systems, related to speed and ease for geo-referencing natural and artificial elements, they suffer several errors such as: atmospheric refraction, multi-path effect, offset instruments, and satellite geometry, among others. As a consequence, there are positional errors in range measurement by code with the L2C and L5 bands, fond on new satellites, with a 95% confidence level of ± 8.5 m (Ghilani *et al*., 2012) without differential correction.

The static GNSS data allows for greater precision in geodesic and topographic surveys. Two receivers are required: one located in a control station with previously determined coordinates with high precision, and the other as receiver at a point whose coordinates are to be determined. The information obtained during the observations is saved in the memory of the receivers, and the differences between the observed coordinates and the fixed ones, gathered from the control station, provide corrections for points whose positions are yet to be determined. The above corresponds to the known method of differential correction in post-process (González, 2009).

A main objective was to evaluate the precision and accuracy of the GNSS positioning in static mode, by comparing vertices of a field calibration pattern, available at a pre-design stage in the Faculty of Engineering of the Universidad del Cauca (located in Popayán, Colombia), in surveys conducted with equipment for the electronic measurement of distances (total station), establishing its suitability for georeferencing surveying networks or initial vertices and azimuth signals for civil engineering projects.

This study revealed the following aspects: the “goodness” of the differential correction for the non-corrected points, the effect of time on the positioning precision, the effect of the test hour, the effect of the type of coordinates used, the shape of the distributions and the linear dependence between the quality variables obtained with the DGNSS method.

Theoretical Framework

A precise positioning is obtained by using high quality GNSS receivers and geodesic antennas. These receivers must be dual frequency in order to mitigate the ionospheric influence and obtain a quick ambiguity fix. In addition, the antennas must be accurately calibrated, to reduce variations of the phase center, and designed to minimize the multi-path effect (Ali *et al*., 2005; Wang *et al*., 2007). The high quality of the geodetic equipment provides positions with high (millimeter level) accuracy with phase measurements because of the resolution of the system (the fractional part of a wavelength can be measured with a precision of 1% of its length, that is 2 mm for L1 band) (Andrei, 2012). This level of accuracy can be achieved in post-processing mode or in real time only after a correct determination of the whole ambiguity (Hofmann *et al*., 2008; Schwieger, 2003).

The procedure to remove or mitigate many of the different sources of errors and improve the accuracy of positioning is to use the principle of differentiation or Differential-GNSS (DGNSS). DGNSS is used for relative positioning where the mobile receiver obtains time-tagged measurements from a base station. In the mobile receiver, the received measurements are differentiated with the corresponding collected measurements. Finally, the mobile receiver estimates its own relative position to the base station by calculating the vector between the two points (Han *et al*., 2012; Rizos, 2003, 2009).

According to Ghilani *et al*. (2012), accuracy is the absolute nearness of a measure in relation to its true value, while precision is the degree of consistency within a group of observations, being evaluated by considering differences between the observed values.

The accuracy of static GNSS positioning, for each point, was evaluated by considering the distance between each static GNSS occupation and the true landmark positions surveyed with the total station. The precision of each measurement is the standard deviation in its horizontal and vertical components, obtained with a software of differential correction procedures (Valbuena *et al*., 2010). The standard deviation of samples from a known set of true values, is called Root Mean Squared Error (RMSE).

Materials and Methods

Study area

The field check for surveying equipment (theodolites, dumpy level and total stations) is located at the Faculty of Engineering of the Universidad del Cauca (Benavides *et al*., 2006). The control points are arranged in a closed traverse perimeter of 696 m over the campus perimeter. It also has an internal traverse perimeter of 252 m around the central park where verification routines can be applied to determine the uncertainty of equipment (**Figure 1**).

The external traverse has alignments with lengths from 47 m to 124 m and relative altitudes between 0.8 m and 5.4 m, arranged in eight vertices. The level of precision obtained by determining the closure error for measurements under repeatability and reproducibility, with a total station traverse, was close to 1:15 000, with a levelling error of 8 mm. The objective of the external traverse is to detect calibration errors in the measurement equipment by performing a full survey with long visuals and using several stations.

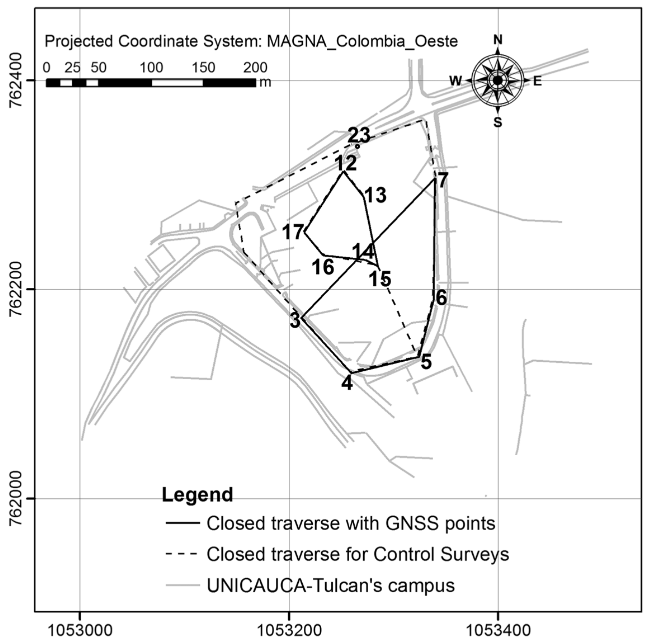

Figure 1. Geometric disposition of the field check and the static GNSS occupation surveys in the study zone.
Source: Authors

The internal traverse contains 6 vertices and consists of alignments between 13 m and 70 m, and relative altitudes between 0,25 m and 2,66 m. The vertical closing error in this traverse is 6 mm; designed for short routines to verify surveying instruments.

Equipment

A Topcon Hiper Lite plus double frequency device, property of the Universidad del Cauca, was used for this study. It has a precision of 3mm + 1ppm in a post-process with static mode. For this study, there were observation times of 10, 30 and 60 minutes.

Work scheme

In the following figure (Figure 2), it is specified the methodological process for this study, corresponding to the recognition and identification of the traverse points, the differential correction of the GNSS positioning into Cartesian coordinates, the exploratory analysis of quality parameters and static GNSS precision, and the evaluation of the distances' errors.

Differential correction

For evaluating the precision of the occupied points, it was applied differential correction of the points' alternative positioning. The computational tool for post-processing, network analysis and adjustment of GNSS data was Topcon Tools 7 (Andrei, 2012), which introduced "tps" files as input data, containing information on the occupations, obtained with dual frequency equipment and the observation and navigation files requested via internet from the POPA station of IGAC. This corresponds to what is technically known as differential correction in post-processing, which reveals the precision of each point position (Hatch *et al*., 1998).

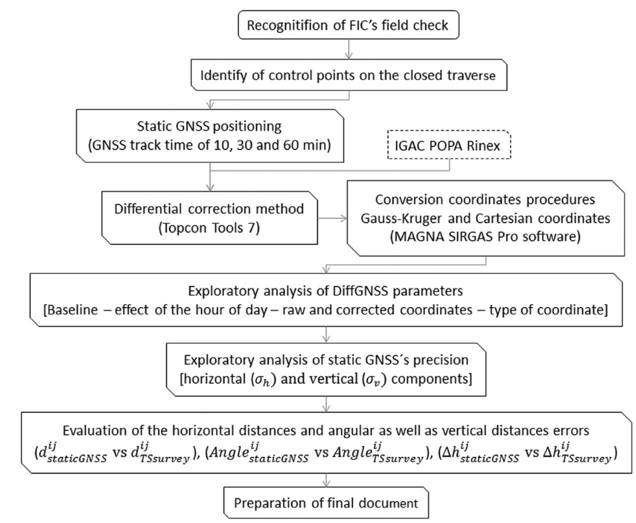

Figure 2. Work scheme.
Source: Authors

Conversion of coordinates

It is important to work with plane coordinates in civil engineering projects: the geometric parameters, such as lengths, are based on Euclidean distances, and so the ellipsoidal coordinates needed to be converted into plane coordinates by using the official Magna SIRGAS software (Martínez *et al*., 2009; Sánchez *et al*., 2009). The sensitizing conditions included:

a. Conversion of geographic coordinates to plane Gauss - Kruger coordinates, using the Magna SIRGAS Pro software.

b. Conversion of geographical ellipsoidal coordinates to plane Cartesian coordinates using the Magna SIRGAS Pro software. The plane Cartesian coordinates are recommended for working at a scale bigger than 1:10 000 (Sánchez *et al*., 2002).

Accuracy of GNSS positions

The pattern of reference available for this stage is the preliminary calibration of topographic equipment of the Universidad del Cauca (Benavides, 2006), where they obtained the horizontal and vertical coordinates of the internal and external traverse's vertices shown in the Figure 1 with the conventional field methodology (total station and dumpy level). During the preliminary design, it was not possible to transfer the coordinates of the state's geodetic network to the study zone, so the location was pinned arbitrarily with the direction indicated by a compass as azimuth signal, and the initial coordinate obtained with a Garmin handheld navigator, which gave the height of the initial point as well. The above, because at the time the GPS-CCI vortex was the only one available with coordinates validated by IGAC (Instituto Geográfico Agustín Codazzi). This condition made it impossible to transfer real coordinates to the study zone that allowed to obtain an initial orientation of the total station, and therefore to compare them with different reference systems, but it was possible to do so in terms of geometric parameters like distances, angles and relative altitudes.

Results and discussion

Goodness of the differential correction

In Figure 3 it can be observed the effect of the differential correction. There, the three uncorrected occupations, pictured with circles (60 minutes), triangles (30 minutes) and rectangles (10 minutes) are arranged at the same point (stars) when corrected.

The range of the horizontal GNSS precision varied between 1 mm and 38 mm, excluding points 14A, 6B, 14B and 17C with horizontal accuracies of 0,33, 0,35, 0,20 and 1,27 m, respectively. The extreme values listed above may have occurred because of the multi path effect: caused in control point 14 because of its covered location, and point 17 because it was near a building. For point 6B with a lapse of one hour, the same cause did not apply (Table 1).

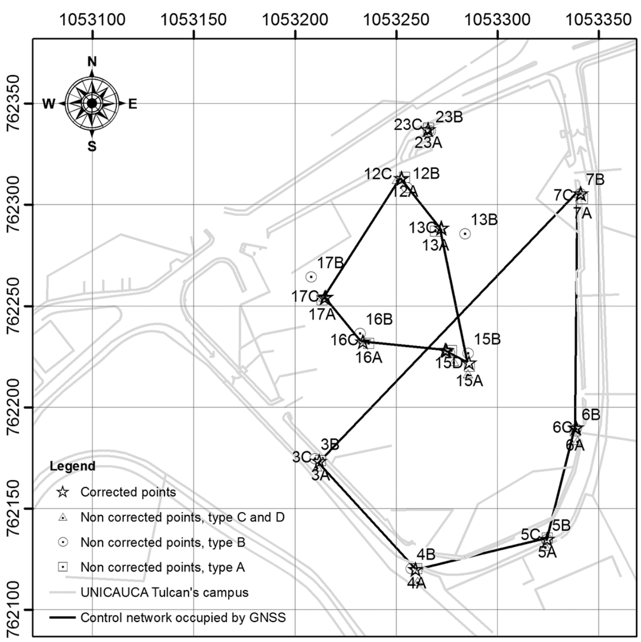

Figure 3 Corrected and original position of the points located with GNSS in three occupation times.
Source: Authors

For the vertical GPS precision, disregarding the same points listed above, there was a variation between 2 mm and 62 mm, showing a relationship twice the horizontal precision (Berber *et al.*, 2012.)

Table 1. Precisions obtained for the occupation times (Root Mean Square in m)

	10 minutes		30 minutes		60 minutes	
Pt	HRMS	VRMS	HRMS	VRMS	HRMS	VRMS
12	0,004	0,011	0,006	0,014	0,010	0,021
13	0,008	0,020	0,012	0,038	0,009	0,020
14	0,017	0,036	0,333	0,166	0,197	0,212
15	0,005	0,013	0,013	0,021	0,001	0,002
16	0,019	0,038	0,008	0,026	0,015	0,039
17	1,267	1,486	0,008	0,019	0,015	0,024
3	0,005	0,008	0,005	0,009	0,009	0,010
4	0,005	0,012	0,009	0,020	0,005	0,010
5	0,026	0,028	0,004	0,007	0,004	0,012
6	0,015	0,024	0,027	0,052	0,353	0,540
7	0,017	0,022	0,013	0,036	0,018	0,056

Source: Authors

The horizontal precision (HRMS) varied between 0,001 m and 1,267 m with an average of 0,68 m and a coefficient of variation of 320,6 %, indicating the effect of outliers in the homogeneity of the variable. However, the 90th percentile corresponded to a value of 0,10 m, indicating that 90 % of the occupied points are more precise than this value. A very similar behaviour was seen in the vertical GPS precision, which varied between 0,002 m and 1,486 m with an average of 0,085 m and a 90th percentile of 0,104 m. The coefficient of bias and kurtosis of the horizontal GPS precision were 4,8 and 22,2, respectively, distant values of a distribution with a Gaussian behaviour. The coefficients of bias and kurtosis of the vertical precision were 4,9 and 26,6; very similar to the values of horizontal precision. An approach to consider the effects of the outliers to metric calculation of precision is seen in Höhle *et al.* (2009) however, need automated filtering and classification in order to generate terrain (bare earth, and, in this particular case, but using single-frequency equipment, in (Feo *et al.*, 2016). The general distribution of errors without outliers shows minor error for the positioning time of 30 minutes (Figure 4). According to Andrei (2012) a similar average behaviour was found in the analysis of base line vectors between 0,5 and 12 km, with occupation times of 25, 31 and 50 minutes.

An unsupervised classification with the Ward method (hierarchical classification) (Lê *et al.*, 2008), can quickly detect occupied points with a different behaviour than the majority (Figure 5).

The hierarchical classification was applied in 4 groups. The outlier was identified as 17C (10 minutes), showing a quite different behaviour than the other groups. A similar behaviour was seen in points 6B, 14A and 14B. The clustering method demonstrated the effect known as multi-path for point 17, located near a facing parameter, where for a time of 10 minutes, the quality of the GPS positioning moved away from the behaviour of other points.

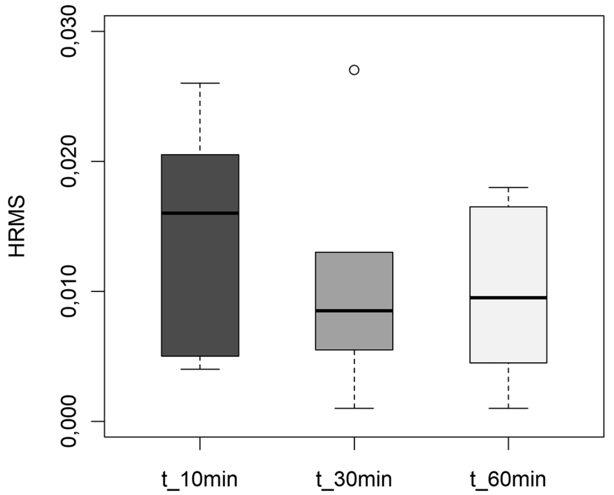

Figure 4. Distribution of the horizontal precision according to the time of occupation.
Source: Authors

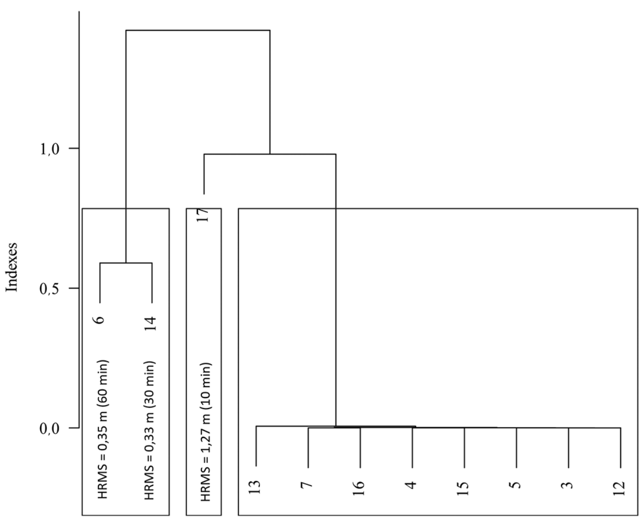

Figure 5. Groups obtained by hierarchical classification.
Source: Authors

Distance to the base

The distances between the control points and the reference station, or POPA base, where it was conducted the differential correction processes, were between 348 and 533 m. This condition is ideal, since the outliers of the baseline vectors are very close; it is assured that the application of the error conditions was similar. Accuracy is worse when the differential correction process is made from more distant base stations, requiring increased occupation times of the observation sessions. There are results for the sensibility analysis for the effect of the baselines and occupation times in Dogan *et al.* (2014), as well as an empirical formula for determining the time range of observation in terms of the length to the baseline in Andrei (2012).

Effect of the hour of day on precision

The database of GNSS precision was grouped in the periods: 7:00 to 11:00, 11:00 to 13:00, 13:00 to 17:00 and 17:00 to 21:00. They were analysed with the statistical technique ANOVA (Table 2).

Table 2. ANOVA analysis of GNSS precision according to the hour of the day

Group	Period		Degrees of freedom	F	p≤	Fcritical value
1	07:00	11:00	7	1.52	0.23	2.9
2	11:00	13:00	14			
3	13:00	17:00	8			
4	17:00	21:00	7			

Source: Authors

Since the probability values (p≤) are higher than the usual level of significance of 5%, it was concluded that there were no significant differences attributed to the hour of day in which occupations were taken with the geodetic GPS equipment.

Accuracy of the occupation points with the GNSS positioning

The accuracy was evaluated by comparing the parameters of distances, internal angles and relative altitudes from the points of the field verification, obtained from the corrected coordinates for different positioning times and from coordinates obtained by three repetitions with Total Station tools.

Uncorrected plain Cartesian coordinates

The lower error in distance was found for a time of 30 minutes, with variations between 0,06 and 2,89 m; 90% of the test points had an error in distance of 2,8 m for the indicated time. In the case of the internal angle, it was found a closer range for the external traverse with a positioning time of 30 minutes, a situation that is not held in the internal traverse where the lower angular error is found for 10 minutes. In any case, there was a global variation of the angular error ranging from 0,3 to 9,93 degrees, which are high values for tolerances in engineering. The ANOVA analysis on the measurements groups gave more significant differences for error in distance ($p \leq 0,08$) than the angular error ($p \leq 0,13$) (Table 3).

Corrected plane Cartesian coordinates

The best distance and angular accuracy in the external traverse (A) was found for the positioning times of 10 and 60 minutes, with minimal errors between 0,001 to 0,014 m in distance and from 0,25 to 1,53 arc minutes in angular. The internal traverse (B) had the best accuracy for positioning times of 60 minutes with minimal errors ranging from 0,007 to 0,091 m and from 2,51 to 8,92 minutes. The RMSE of the distance was 1 cm in the external traverse and 5 cm in the internal one; the angular RMSE was 1′7 ″ in the external traverse and 6′8″ in the internal traverse.

Table 3. Range of accuracy obtained in distance and angles (non-corrected coordinates)

Traverse	Duration for observing sessions (min)	Distance range (m)	Angle range (degrees)
A	10	0,31 to 3,80	0,30 to 4,70
	30	0,06 to 2,14	0,30 to 3,70
	60	0,14 to 1,64	0,20 to 2,90
B	10	0,14 to 5,08	1,30 to 9,90
	30	0,07 to 2,89	1,70 to 14,0
	60	4,13 to 8,89	3,80 to 34,0

A: External traverse – B: Internal traverse.
Source: Authors

This shows that an increased positioning time is necessary for achieving adequate accuracy when the points are subjected to multipath error. Otherwise, 10 minutes is good enough to achieve accuracy with a value of 10 cm. corresponding to 90% of all points analyzed for the occupancy time indicated. (Table 4). This result is similar to the findings of Andrei (2010)PPP can be considered as an efficient alternative to the conventional differential positioning methods. This paper describes an analysis of PPP performance from the accuracy, precision, convergence period, and availability point of view. International GNSS Service (IGS.

Table 4. Range of accuracy obtained in distance and angles (corrected coordinates)

Traverse	Duration for observing sessions (min)	Distance range (m)	Angle range (minutes)
A	10	0,001 to 0,014	0,25 to 1,53
	30	0,003 to 0,239	0,76 to 17,50
	60	0,014 to 0,036	0,068 to 4,472
B	10	0,012 to 0,726	1,59 to 94,02
	30	0,020 to 0,547	0,61 to 31,45
	60	0,007 to 0,091	2,511 to 8,917

Source: Authors

By comparing the distances and directions of the internal and external traverse, obtained with the static GNSS method and using the same geometric characteristics gathered from a more accurate source of topographic data (total station), the RMSE was calculated. The result showed that, when the points of the traverse (external traversal) are not subjected to the multipath effect, a shorter occupation time in the observation session is required (Table 5).

Table 5. Results of the RMSE

Traverse	Duration for observing sessions (min)	RMSE in Distance (m)	RMSE in angle (minutes)
A	10	0,01	1,12
	30	0,15	12,44
	60	0,02	2,74
B	10	0,33	59,40
	30	0,34	17,77
	60	0,05	6,14

Source: Authors

Plane Gauss Kruger Gaussian Coordinates (corrected)

With this type of coordinates, the external traverse (A) had the best accuracy in distance for positioning times of 30 minutes, while the angular accuracy had a good performance for positioning times of 10 and 60 minutes. In the internal traverse (B), the accuracy in distance was better for both 10 and 60 minutes, but the best angular accuracy was achieved at 60 minutes. Considering that the 90% of the accuracies in the global distances was 69 cm, it was found that the Gaussian plane coordinates were not appropriate for ensuring tolerance levels in engineering. The best results in the sensitizing of the coordinate type was achieved with plane Cartesian coordinates, according to the guidelines for implementing the MAGNA-SIRGAS system as official datum for Colombia (IGAC. 2004).

Accuracy of height differences (vertical distances)

For vertical distances, the comparison was based on the corrected ellipsoidal height for each occupied point, and calculating the geoid undulation for each site. Then, estimated the physical height obtained with GPS, which was compared to the relative altitude data gathered with a total station.

According to the results of the internal traverse, in the case of relative altitudes, a direct relationship between time and positioning accuracy of the vertical distance was established. A longer positioning time implies better proximity to the reference data. In the case of the absolute error of the relative altitudes of the external traverse, the minimum RMSE obtained was 0.18 m, for occupation time 60 minutes. In the case of the internal traverse, the minimum RMSE applied for an occupation time of 30 minutes was 0.15 m (Table 6). These results are similar to the study of Yuan, Fu, Sun, & Toth (2009), who indicated that the tolerance for checkpoint coordinates must be less than 0,25 m for planimetry and less than 0.30 m for elevation in plane surveying at a scale of 1:500.

Table 6. RMSE for relative altitudes

Traverse	Cosecutive line	Absolute error			RMSE		
		10 min	30 min	60 min	10 min	30 min	60 min
A	3 - 4	4.98	0.03	0.05	3,52	0,26	0,18
	4 - 5	4.98	0.02	0.08			
	5 - 6	0.10	0.40	0.26			
	6 - 7	0.01	0.03	0.21			
	7 - 3	0.11	0.33	0.08			
B	15 – 14	0.15	0.25	0.27	3,19	0,15	0,22
	14 – 16	4.78	0.22	0.39			
	16 – 17	1.24	0.00	0.04			
	17 – 12	1.28	0.04	0.04			
	12 – 13	0.08	0.03	0.04			
	13 - 15	4.98	0.03	0.00			

Source: Authors

Precision-accuracy ratio

There are direct relationships between the precision and accuracy for occupancy times of 10 and 30 minutes, but not for an occupation time of 60 minutes (**Figure 6**), as seen in Valbuena *et al*. (2010). In that study, it was concluded that high values of the absolute error are not always related to high values of precision, suggesting that the former is the best descriptor of GNSS positioning performance.

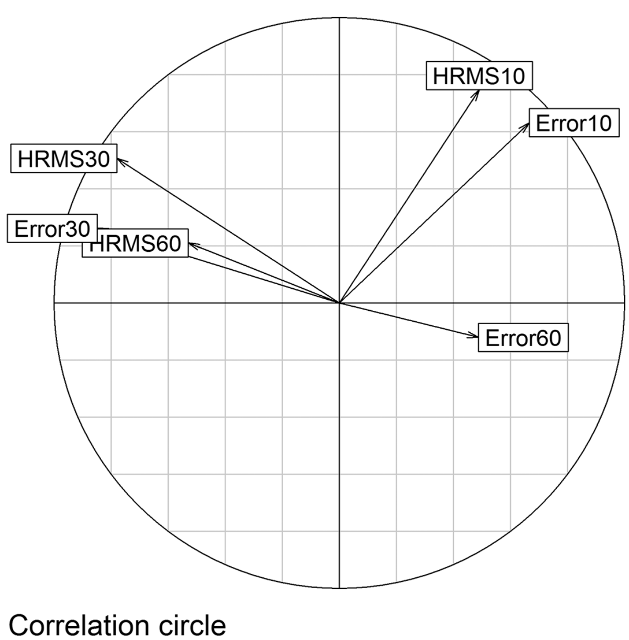

Figure 6. Precision-accuracy ratio obtained by Principal Components Analysis.
Source: Authors

Conclusions

The precision of the differential corrected data had a direct impact on its accuracy. This process is needed in the GPS survey procedure to quantify uncertainty on the occupation points. It is important to be able to compare the results to a national reference system.

In this study, it was demonstrated that plain Cartesian coordinates must be used to achieve tolerable errors in engineering applications. In addition, since there is no direct relation between better precision and larger occupation time, and the hour of the day had no relevant effect, it is better to consider the multipath effect to guaranty the quality of control points with static GNSS positioning.

According to the RMSE found in the horizontal and vertical distances, the detail level obtained was equivalent to a scale of 1:500, meeting the requirements for spatial data in engineering. On the other hand, the angular error did not satisfy the tolerance for high accuracy surveys, probably because the comparison was made in relative terms and not absolute ones.

This study only considered low baselines to the base station; the effect of large baselines on the precision and accuracy should be evaluated in additional studies, as well as the optimal time to obtain better precision.

References

Ali, R., Cross, P., & El-sharkawy, A. (2005). *High Accuracy Real-time Dam Monitoring Using Low-cost GPS Equipment High Accuracy Real-time Dam Monitoring Using Low-cost GPS Equipment*. In FIG Working Week.

Andrei, C.-O. (2010). *Performance analysis of the precise point positioning*. Journal of Geodesy and Cadastre, 9–20.

Andrei, C. O. (2012). *Cost-effective precise positioning using carrier phase navigation-grade receiver*. In *2012* International Conference on Localization and GNSS, *ICL-GNSS 2012*. DOI:10.1109/ICL-GNSS.2012.6253132

Benavides, M., Plaza, G., & Correa, N. (2006). *Metrología para Equipos Topográficos de Precisión*. Revista Ingeniería *Hoy.*, *25*, 69–76.

Berber, M., Ustun, A., & Yetkin, M. (2012). *Comparison of accuracy of GPS techniques. Measurement:* Journal of the International Measurement Confederation, *45*(7), 1742–1746. DOI:10.1016/j.measurement.2012.04.010

Bernabé, M., & López, C. (2012). *Fundamentos de las Infraestructuras de Datos Espaciales (IDE)*. Madrid: Universidad Politécnica de Madrid.

Dogan, U., Uludag, M., & Demir, D. O. (2014). *Investigation of GPS positioning accuracy during the seasonal variation*. Measurement, *53*, 91–100. DOI:10.1016/j.measurement.2014.03.034

Feo, C. A. M., Martinez, L. J. M., & Muñoz, N. A. C. *Accuracy and spatial variability in GPS surveying for landslide mapping on road inventories at a semi-detalled scale: The case in Colombia,* 41International Archives of the Photogrammetry, Remote Sensing and Spatial Information Sciences - ISPRS Archives 291–297 (2016). Praga: ISPRS - International Archives of the Photogrammetry, Remote Sensing and Spatial Information Sciences. DOI:10.5194/isprsarchives-XLI-B2-291-2016

Ghilani, C. D., & Wolf, P. R. (2012). *Elementary Surveying: An Introduction to Geomatics (13th Edition)*. *Vasa*. Prentice Hall. Retrieved from http://medcontent.metapress.com/index/A65RM03P4874243N.pdf%5Cnhttp://www.getcited.org/pub/102714699

González, A. (2009). *Sistema de Posicionamiento Global (GPS)*. Editorial Escuela Colombiana de Ingeniería.

Han, J.-Y., Wu, Y., & Liu, R.-Y. (2012). *Determining the optimal site location of GNSS base stations*. Boletim de Ciencias Geodésicas, *18*(1), 154–169.

Hatch, R. R., & Nelson, F. (1998). *Method and system for post-processing differential global positioning system satellite positional data.* Google Patents.

Hofmann, B., Lichtenegger, H., & Wasle, E. (2008). *Global Navigation Satellite Systems: GPS, GLONASS, Galileo and more*. Wien, Austria: Springer-Verlag Berlin Heidelberg.

Höhle, J., & Höhle, M. (2009). *Accuracy assessment of digital elevation models by means of robust statistical methods.* ISPRS Journal of Photogrammetry and Remote Sensing, *64*(4), 398–406. DOI:10.1016/j.isprsjprs.2009.02.003

IGAC. (2004). *Aspectos prácticos de la adopción de marco geocéntrico nacional de referencia Magna-Sirgas como datum oficial de Colombia*.

Jackson, D. D., & Kagan, Y. Y. (2011). *Characteristic Earthquakes and Seismic Gaps.* In Encyclopedia of Solid Earth Geophysics (Vol. 5, p. 1539). DOI:10.1007/978-90-481-8702-7

Lê, S., Josse, J., & Husson, F. (2008). *FactoMineR : An R package for multivariate analysis.* J Stat Softw, *25*, 1–18.

Martínez, W. A., & Sánchez, L. (2009). *Realization of the SIRGAS Reference Frame in Colombia*. In Geodetic Reference Frames (pp. 185–190). Springer.

Rizos, C. (2009). Network RTK *Research and Implementation-A Geodetic Perspecti.* Positioning, *1*(2).

Rizos, C., & Han, S. (2003). R*eference station network based RTK systems-concepts and progress*. Wuhan University Journal of Natural Sciences, *8*(2), 566–574.

Sánchez, L., & Brunini, C. (2009). *Achievements and Challenges of SIRGAS*. I*n Geodetic reference frames* (pp. 161–166). Springer.

Sánchez, L., & Martínez, W. (2002). *Approach to the new vertical reference system for Colombia*. *In Vertical Reference Systems* (pp. 27–33). Springer.

Schwieger, V. (2003). *Using Handheld GPS Receivers for Precise Positioning.* In Proceedings of the 2nd FIG Regional Conference. Marrakech, Morocco.

Valbuena, R., Mauro, F., Rodriguez-Solano, R., & Manzanera, J. a. (2010). *Accuracy and precision of GPS receivers under forest canopies in a mountainous environment.* Spanish Journal of Agricultural Research, *8*(4), 1047–1057.

Wang, C., & Xiao, L. (2007). *Sensor localization under limited measurement capabilities.* IEEE Network, *21*(3), 16–23.

Yuan, X., Fu, J., Sun, H., & Toth, C. (2009). *The application of GPS precise point positioning technology in aerial triangulation*. ISPRS Journal of Photogrammetry and Remote Sensing, *64*(6), 541–550. DOI:10.1016/j.isprsjprs.2009.03.006

Zhang, L., Mao, Q., Li, Q., & Zhang, P. (2014). *An accuracy-improvement method for GPS/INS kinematic levelling for use in linear engineering surveying projects*. Measurement, *54*, 22–30. DOI:10.1016/j.measurement.2014.03.026