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Revista Colombiana de Tecnologías de Avanzada
UNIPAMPLONA

Digital Object Identifier: 10.24054/rcta.v1i45.3025

Comparative technical study of topographic surveys in mountainous areas carried out with total station, GNSS equipment, and non-specialized UAV with and without control point support

Estudio técnico comparativo de levantamientos topográficos en terreno montañoso realizados con estación total, equipos GNSS, y UAV de consumo con y sin apoyo de puntos de control

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Received: *july 29, 2024.* ***Accepted:*** *december 11, 2024.* ***Published:*** *january 01, 2025.*

How to cite: *S. B. Chaves Pabón and E. F. Muñoz Serna, "Comparative technical study of topographic surveys in mountainous areas carried out with total station, GNSS equipment, and non-specialized UAV with and without control point support", RCTA, vol. 1, no. 45, pp. 19–31, jan. 2025.*
Recovered from https://ojs.unipamplona.edu.co/index.php/rcta/article/view/3025

Abstract: This study compares three commonly used topographic technologies for mapping mountainous terrain: total station, GNSS equipment, and non-specialized UAV (with and without control points). The comparison considered chronological and technical criteria to determine the advantages and disadvantages of each technology. The study found that GNSS antennas, when they have optimal satellite reception and communication between the base and rover, offer the best performance compared to the other survey methods. They achieved accuracy similar to that of a total station, with considerably shorter execution times. Another finding was that the data collected with a UAV using control points allowed for a geomorphological description of the area but with significantly lower accuracy than the other technologies. These results emphasize the importance of performing a high densification of control points to achieve greater accuracy, even if it means increasing the required fieldwork time.

Keywords: drone surveying, altimetry topographic survey, contour lines, digital elevation model "MDE", real-time kinematic "RTK".

Resumen: El objetivo del presente trabajo es realizar una comparación de tres tecnologías topográficas usadas comúnmente en el mapeo de terrenos montañosos: estación total, equipos GNSS y UAV de consumo. En esta comparación tomamos en cuenta criterios cronológicos, y técnicos para determinar las ventajas y desventajas de cada una de las tecnologías analizadas. En este estudio se determinó que las antenas GNSS, siempre y cuando cuenten con recepción satelital y una comunicación entre base y rover óptimas, ofrecen el mejor desempeño comparadas con las otras dos. Estas obtuvieron una precisión similar a la conseguida con estación total, con tiempos de ejecución considerablemente menores. En segundo lugar, se encontró que los datos recolectados con UAV, con puntos de control, permitieron realizar una descripción geomorfológica de la zona cuya precisión es sensiblemente inferior a la conseguida con las otras dos tecnologías analizadas. Esto evidenció que, para conseguir mayor precisión, es necesario realizar una alta densificación de puntos de control, aun cuando esto suponga un aumento de los tiempos de trabajo en campo.

Palabras clave: topografía con drones, altimetría, líneas de contorno, modelo digital de elevación “MDE”, navegación cinética satelital en tiempo real “RTK”.

1. INTRODUCTION

The use of the global navigation satellite system (GNSS) and UAVs (Unmanned Aerial Vehicle) [1] has seen a significant increase in recent years, marking a substantial shift in the field of surveying. These technological advances, while relatively new, have quickly gained traction, offering unique advantages over the traditional total station. However, the total station, a combination of an electronic theodolite and distance meter [4] [5], remains the most widely used instrument in surveying, particularly in applications demanding the highest possible accuracy.

Although the total station is considered the leading equipment in topography surveying due to its reliability and accuracy, the technology has two main disadvantages. Firstly, the need to maintain visual contact between the point of assembly and the points to be taken leads to relatively high survey times and costs. Secondly, in some cases, difficulty accessing terrain areas where points need to be taken may pose a risk to surveying team members or even make obtaining information from that area impossible. On the other hand, obstacles in the terrain, such as trees, tall vegetation, power lines, mountainous areas, or unfavorable weather, can lead to data degradation obtained with GNSS and UAV [2]. Additionally, measurements taken with GNSS equipment can be affected by distance, leading to decreased accuracy as the distance increases [3].

The Global Navigation Satellite Systems (GNSS) use a network of satellites to provide geo-location positioning. The most well-known system is the GPS, which is administered by the US military, although there are other alternatives, such as the Galileo system (European Union), GLONASS (Russia), and Beidou (China) [6]. This study will focus on GNSS receivers for topography, which have high-precision receivers capable of obtaining measurements below a centimeter.
Various positioning methods are associated with this technology, such as NTRIP or RTK. In this study, the real-time kinematic (RTK) method has been used to achieve the best positioning accuracy possible, combined with the GPS, Galileo, GLONASS, and Beidou systems simultaneously. Ultimately, the highest positioning accuracy is achieved using RTK combined with the GPS and GLONASS satellite systems [7].

In their study on UAV photogrammetry accuracy, Ferrer Gonzales (et al.) underscore the importance of photogrammetric inputs, such as orthophotos, digital surface models, and elevation models, to describe landscape morphology [8]. Given these needs, the introduction of UAVs in the field of surveying has been a transformative event, becoming an option for image capture for photogrammetric purposes at low cost. Nowadays, not only specialized fixed-wing UAVs or multi-rotors with professional cameras and sub-meter GNSS sensors with RTK technology are used to perform photogrammetry, but non-specialized drones have also become increasingly popular among surveyors [9]. Non-specialized UAVs have enabled small companies or independent surveyors to access these photogrammetric inputs.

However, it is essential to determine the accuracy levels of photogrammetric products obtained with non-specialized UAVs in areas with significant elevation differences and constant slope changes. One way to improve photogrammetric products' absolute and relative accuracy is by establishing georeferenced aerial control points using precise surveying equipment [10]. These points are marked on the ground so that the image-processing software can identify them. While using control points significantly increases the time required for fieldwork, it also allows for greater accuracy. Therefore, this study also aims to analyze the

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advantages and disadvantages associated with using control points when utilizing UAV technology.
It is essential to assess each surveying method’s accuracy and time requirements when conducting topographic characterization in mountainous areas. This information can guide the selection of the most suitable method for the specific job requirements. However, existing research only compares two technologies or does not consider the unique geographic conditions of mountainous terrain [3] [11]. Our research aims to conduct a comparative analysis of the traditional surveying method (total station), GNSS antennas for topography, and non-specialized UAVs (with and without control points) to address this gap in the literature. The time required for topographic surveys and the technical aspects associated with each technology will be evaluated.

2. METHODOLOGY

This section describes the conditions under which this study was carried out. It describes the specific area that has been measured, the technical tools and surveying technologies that have been used, as well as the methods employed for data processing and analysis.

2.1. Study Area

For our research, A topographical analysis of the same geographical area has been carried out using four different survey methods. The land selected for this study is located in the urban area of the municipality of Neira Caldas, Colombia. It covers an area of 39,900 m² and ranges in altitude from 1780 to 1882 meters above sea level. This geographical area has been selected because of the mountainous geography typical of the Andean region of central Colombia.

2.2. Equipment

The equipment used to compare the four survey methods that were analyzed in this study were as follows:

Total station: Trimble M3 DR station, which has an angular accuracy of 2".

GNSS receivers: Tersus GNSS antennas, Oscar Basic model, multifrequency, and multi-constellation. One of the advantages of the RTK method is its speed in capturing information. However, in order to achieve such accuracy, there must be an optimal connection with the base antenna since this is the one that sends the necessary corrections so that the Rover can achieve the required accuracy.

UAV: In this research, a non-specialized Autel Evo 2 UAV equipped with a 48 megapixel camera and a non-specialized GNSS receiver has been used. To survey the terrain, a 15-minute flight plan was developed. The UAV captured 289 images throughout this flight, as illustrated in Figure 1.

Fig. 1. *Flight plan.* ***Source****: own elaboration.*

PC: The equipment used in the office work phase was a PC with an AMD Ryzen 7 5800x processor, 32 GB of RAM and an Nvidia Geforce RTX3070 graphics card.

2.3. Cartographic Projection

To represent the Earth’s curved surface on a flat map, a projection system has been needed that can convert geographic coordinates expressed angularly (latitude and longitude) into two-dimensional coordinates on a plane with X and Y axes (East and North). Since 2020, Colombia has used the Single Origin projection (Origen Único) as the official standard. However, for our study, the Magna Sirgas Central Origin projection, which was developed before the Single Origin, has been chosen. This projection has been selected because it has a scale factor of 1, while the Single Origin has a scale factor of 0.9992, which can cause distortions in photogrammetric processing and GNSS data acquisition. Since our main goal is to analyze the accuracy of each technology, the projection system that would give the most accurate results has been selected.

2.4. Base Topographic Points

Two reference points were established to monitor the chosen area. Firstly, the GPS1 point was installed using the GNSS base antenna and the NTRIP network connection of the National Geological Centre (Centro Geológico Nacional). The coordinates were captured over 10 minutes with a root-mean-square error of 0.075 meters in the horizontal component and 0.123 meters in the vertical component. The second point, DTA1, was demarcated using the Rover antenna with the RTK method, with a root-mean-square error in the horizontal component of 0.009 meters (Eq1), and a vertical error of 0.028 meters (Eq2).

$$RMSE = \sqrt{\frac{1}{n}\sum_{n-1}^{n}((x_i - x_{ref})^2 + (y_i - y_{ref})^2)/2} \quad (1)$$

where: x_i and y_i represent any readings taken; x_{ref} and y_{ref} are the references or real values; n is the number of observations. Therefore: $(x_i - x_{ref})^2 + (y_i - y_{ref})^2$ is the observational residual error.

$$VRMSE = \sqrt{\frac{1}{n}\sum_{n-1}^{n}(z_i - z_{ref})^2} \quad (2)$$

where: z_i represent any readings taken; z_{ref} and y_{ref} are the references or real values; n is the number of observations. Therefore: $(z_i - z_{ref})^2$ is the observational residual error.

After setting the GPS1 and DTA1 coordinate points, topographic points were set using the total station and GNSS equipment, and aerial control points were established, as shown in Table 1.

Table 1: *Base Topographic Points*

GNSS Base Points			
Point	**North**	**East**	**Height**
GPS 1	1064563.999	839831.503	1881.122
DTA1	1064549.182	839849.410	1881.116

Source*: own elaboration.*

2.5. Field Work

Fieldwork was carried out over three consecutive days. The first day had clear skies, which provided optimal visibility, while the second and third days were cloudy. Throughout this time, there were strong gusts of wind, reaching speeds of up to 40 km/h, which significantly influenced our work.

Once we georeferenced the two points using the GNSS equipment, we set the total station at DTA1, pointing to GPS1 to orient the station. Then, we draw a 5-delta polygonal, as shown in Figure 2. Now, it should be noted that the difference in distances between the deltas was due to the difficulty of obtaining a visual at some points, given the unevenness of the terrain.

Fig. 2*. Polygonal.* ***Source****: own elaboration.*

The closing error was 0.042m (Eq3), for an accuracy of 1/9000 (Eq4).

$$E_{cierre} = \sqrt{(E_y)^2 + (E_x)^2} \quad (3)$$

where E_y is the error in the y-component, and E_x the error in the X component

$$Precisión = \frac{E_{cierre}}{Distancia\ poligonal} \quad (4)$$

2.6. Data Collection

The topographic control points, referenced by their North, East, and Height coordinates, form the basis for characterizing the area we surveyed. To achieve this, we established a grid covering the entire area of study, focusing specifically on changes in elevation, slopes, edges, and points of interest. The method we employed for taking measurements involved the use of total station and GNSS antennas, and it is detailed below. It is crucial to note that we categorized all the points we measured into three groups: ground points, control points, and aerial control points.

2.6.1. Ground Points

We included in this group the points we used for terrain characterization. We set these points to create a gridline that covers the study area, paying special attention to changes in height or slope. In this work we set 408 terrain points.

2.6.2. Control Points

We used control points to make a precise comparison of the four methods we analyzed in this study. These points were marked in the field and georeferenced using a total station and GNSS antennas. We established a total of 10 control points, which are listed in Table 2 and shown in Figure 3. It is important to note that we did not include these points when creating the surfaces in the AutoCAD Civil 3D software (educational license).

Table 2: *Control points*

Control Points			
Point	**North**	**East**	**Height**
CTRL1	1064545.590	839833.535	1881.391
CTRL2	1064556.667	839772.390	1850.264
CTRL3	1064534.666	839694.294	1837.782
CTRL4	1064563.307	839728.071	1837.472
CTRL5	1064585.928	839689.994	1831.359
CTRL6	1064598.032	839645.704	1820.271
CTRL7	1064589.123	839873.554	1871.636
CTRL8	1064513.791	839758.638	1860.973
CTRL9	1064507.689	839815.690	1882.067
CTRL10	1064451.462	839788.050	1869.692

Source: *own elaboration.*

Fig. 3. *Control points.* ***Source:*** *own elaboration.*

2.6.3. Aerial Control Points

Aerial control points are essential for adjusting mosaics during photogrammetric processing. In this study, we placed 5 aerial control points, detailed in Table 3 and Figure 4.

Table 3: *Aerial Control Points*

Aereal Control Points			
Point	**North**	**East**	**Height**
35	1064523.786	839817.402	1882.072
76	1064594.233	839849.307	1865.781
122	1064533.855	839701.778	1837.869
254	1064567.610	839615.542	1797.492
387	1064453.418	839806.999	1869.536

Source: *own elaboration.*

Fig. 4. *Aerial control points.* ***Source:*** *own elaboration.*

2.7. Processing

2.7.1. UAV Processing

In the image processing phase, we first automatically oriented the photographs by identifying key points. Then, we made necessary adjustments based on the aerial control points set in the field. This process involved placing markers at the points where each mark was located in the photographs. Next, we calibrated the colors and white balance, created a point cloud, and added color using information from the photographs. We then classified the points, and after the image processing software finished, we reviewed the mosaic to identify and correct any potential errors in the classification. After that, we created the mesh, masks, texturing, and tiled model.

Then, we generated a digital elevation model, an orthomosaic, and contour lines with a one-meter difference using these models. We chose this distance for drawing the contour lines to avoid conflicts in the creation of the surfaces, which would have hindered comparative analysis. Finally, we used AutoCAD Civil 3D software to upload the contour lines and create a surface from them, which is visible in Figure 5.

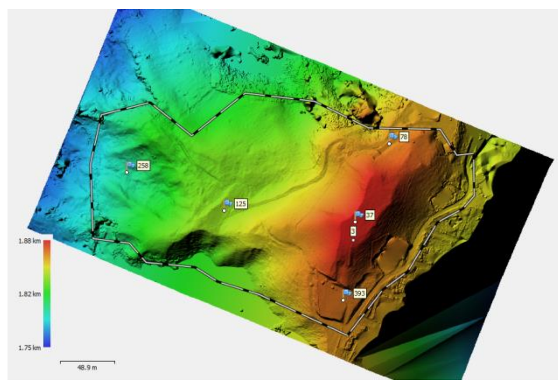

Fig. 5. *Digital elevation model photogrammetric software.* ***Source:*** *own elaboration.*

It is important to note that we conducted two image processing processes. The main distinction between them is that one was conducted without adjustments using aerial control points, while the other included this procedure.

2.7.2. GNSS Antennas & Total Station Processing

The image processing process for GNSS antennas and total stations was similar to that for UAVs. In both cases, we generated a point cloud map using the field data and created a contour lines surface using AutoCAD Civil 3D software. Figure 6 displays the rainbow model produced by AutoCAD Civil 3D using the data collected with the total station.

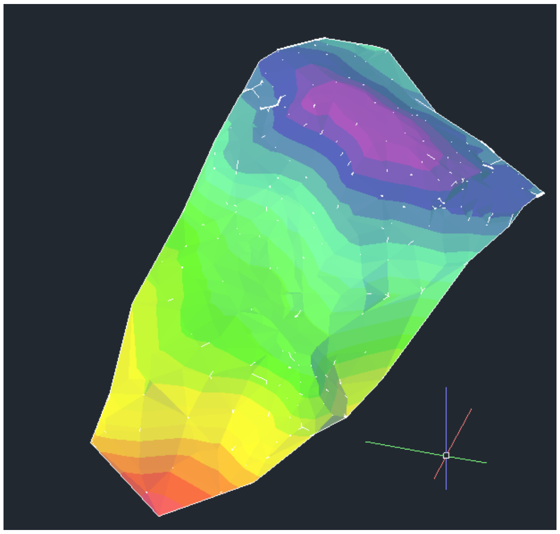

Fig. 6. *Rainbow model of the study area according to total station data.* ***Source:*** *own elaboration.*

3. PRECISION ANALYSIS

3.1. Comparison of Points Measured Directly in the Field

When we compared the results obtained using the four different methods for this study, we discovered that the surface created from the data obtained with the UAV, without control points, had an offset of more than 3 meters in the horizontal component and more than 5 meters in the vertical component. To address this issue, we decided to re-generate the contour lines by shifting the entire dataset using the coordinates from point CTRL9 as a reference, which we obtained using the UAV with control points method. This adjustment improved the absolute precision of this method, allowing us to compare its relative precision with the other methods. Using the control points, we created several comparative tables describing the differences between each dataset in their Cartesian components and the root mean square error in the vertical and horizontal components. Table 4 compares the points we set in the field and the coordinates obtained from the UAV model with control points in the photogrammetric software model space.

Table 4: *Comparison of Control Points: Total Station & GNSS Antennas*

TOTAL STATION				GNSS ANTENNAS				TOTAL STATION / GNSS ANTENNAS					
Point	**North**	**East**	**Height**	**North**	**East**	**Height**	**DN**	**DE**	**DH**	**DN ABS**	**DE ABS**	**DZ ABS**	**HRMS**
CTRL1	1064545.590	839833.535	1881.391	1064545.581	839833.527	1881.414	0.009	0.008	-0.023	0.009	0.008	0.023	0.008
CTRL2	1064556.667	839772.390	1850.264	1064556.652	839772.396	1850.278	0.015	-0.006	-0.014	0.015	0.006	0.014	0.012
CTRL3	1064534.666	839694.294	1837.782	1064534.650	839694.289	1837.803	0.016	0.005	-0.021	0.016	0.005	0.021	0.012
CTRL4	1064563.307	839728.071	1837.472	1064563.306	839728.065	1837.482	0.001	0.006	-0.010	0.001	0.006	0.010	0.004
CTRL5	1064585.928	839689.994	1831.359	1064585.914	839689.969	1831.346	0.014	0.025	0.013	0.014	0.025	0.013	0.020
CTRL6	1064598.032	839645.704	1820.271	1064598.039	839645.701	1820.287	-0.007	0.003	-0.015	0.007	0.003	0.015	0.005
CTRL7	1064589.123	839873.554	1871.636	1064589.135	839873.566	1871.618	-0.012	-0.012	0.018	0.012	0.012	0.018	0.012
CTRL8	1064513.791	839758.638	1860.973	1064513.777	839758.633	1860.975	0.014	0.005	-0.002	0.014	0.005	0.002	0.011
CTRL9	1064507.689	839815.690	1882.067	1064507.695	839815.695	1882.082	-0.006	-0.005	-0.014	0.006	0.005	0.014	0.006
CTRL10	1064451.462	839788.050	1869.692	1064451.435	839788.061	1869.715	0.027	-0.010	-0.023	0.027	0.010	0.023	0.020
					Average		0.007	0.002	-0.009	0.012	0.008	0.015	0.011

Source: *own elaboration.*

3.2. Comparison of Total Station & UAV With Control Points

The first comparison shows a relatively low difference between the North, East, and Elevation components we gather with the total station and the GNSS equipment. The average discrepancies between the two methods are 0.012 meters in the north component, 0.008 meters in the east component, and 0.015 meters in elevation. The horizontal mean square error averages out to be 0.011 meters.

When we compared the information gathered with the total station and the UAV, using control points, we found a significant discrepancy in all the items analyzed, as detailed in Table 5. The average differences revealed an offset of 0.134 meters in the north component, -0.095 meters in the east component, and 0.185 meters in height. These findings indicate that there is no general vertical or horizontal displacement of the UAV set with control points, but rather discrepancies across the entire data set.

Table 5: *Comparison of Control points: Total Station & UAV with control points*

TOTAL STATION				UAV WITH CONTROL POINTS			TOTAL STATION/UAV WITH CONTROL POINTS						
Point	North	East	Height	North	East	Height	DN	DE	DH	DN ABS	DE ABS	DZ ABS	HRMS
CTRL1	1064545.590	839833.535	1881.391	1064544.98	839833.926	1882.836	0.608	-0.391	-1.445	0.608	0.391	1.445	0.511
CTRL2	1064556.667	839772.390	1850.264	1064556.48	839772.093	1849.155	0.183	0.297	1.109	0.183	0.297	1.109	0.247
CTRL3	1064534.666	839694.294	1837.782	1064534.88	839694.469	1837.205	-0.215	-0.175	0.577	0.215	0.175	0.577	0.196
CTRL4	1064563.307	839728.071	1837.472	1064563.24	839727.762	1836.114	0.062	0.309	1.358	0.062	0.309	1.358	0.223
CTRL5	1064585.928	839689.994	1831.359	1064585.65	839689.813	1830.779	0.277	0.181	0.580	0.277	0.181	0.580	0.234
CTRL6	1064598.032	839645.704	1820.271	1064597.76	839645.568	1819.716	0.271	0.136	0.555	0.271	0.136	0.555	0.214
CTRL7	1064589.123	839873.554	1871.636	1064588.3	839872.83	1870.671	0.822	0.724	0.965	0.822	0.724	0.965	0.775
CTRL8	1064513.791	839758.638	1860.973	1064513.88	839759.125	1861.426	-0.086	-0.487	-0.453	0.086	0.487	0.453	0.350
CTRL9	1064507.689	839815.690	1882.067	1064507.52	839816.471	1883.605	0.173	-0.781	-1.538	0.173	0.781	1.538	0.566
CTRL10	1064451.462	839788.050	1869.692	1064452.22	839788.818	1869.544	-0.758	-0.768	0.148	0.758	0.768	0.148	0.763
					Average		0.134	-0.095	0.186	0.346	0.425	0.873	0.408

Source*: own elaboration.*

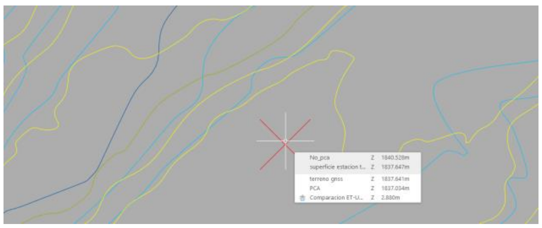

Fig. 7. *Comparison of heights over surfaces. Source: own elaboration.*

We compared the terrain elevations on the surfaces generated by AutoCAD Civil 3D software. The baseline for these surfaces was created using the coordinates of the control points obtained using a total station, as illustrated in Figure 7.

In this case, we included the model generated with the information collected by the UAV without aerial control points.

Table 6 shows that the points taken on the surface using the total station have an average discrepancy of 0.247 meters, and the GNSS antennas yielded similar results with a discrepancy of 0.258 meters. When employing aerial control points, the UAV method reduced the height discrepancy to 0.777 meters compared to the total station data. Conversely, the UAV method without control points had a discrepancy of 1.844 meters, the highest among all the methods, surpassing the others by a significant margin.

Table 6: *Comparison of control points taken in the field with total station and points checked on the surfaces*

POINT COLLECTED IN THE FIELD				POINT OBTAINED IN POINT OBTAINED ON THE AUTOCAD CIVIL 3D SURFACE											
TOTAL STATION				T.E	GNSS	UAV PCA	UAV NO PCA	DIF E.T	DIF E.T (ABS)	DIF GNSS	DIF GNSS (ABS)	DIF UAV PCA	DIF UAV PCA (ABS)	DIF UAV NO PCA	DIF UAV NO PCA (ABS)
Point	North	East	H	H	H	H	H	H	H	H	H	H	H	H	H
CTRL1	1064545.590	839833.535	1881.391	1881.472	1881.479	1881.856	1882.323	-0.081	0.081	-0.088	0.088	-0.465	0.465	-0.932	0.932
CTRL2	1064556.667	839772.390	1850.264	1850.581	1850.668	1849.250	1851.883	-0.317	0.317	-0.404	0.404	1.014	1.014	-1.619	1.619
CTRL3	1064534.666	839694.294	1837.782	1837.647	1837.641	1837.034	1840.528	0.135	0.135	0.141	0.141	0.748	0.748	-2.746	2.746
CTRL4	1064563.307	839728.071	1837.472	1837.251	1837.260	1836.121	1839.828	0.221	0.221	0.212	0.212	1.351	1.351	-2.356	2.356
CTRL5	1064585.928	839689.994	1831.359	1831.117	1831.108	1830.712	1833.341	0.242	0.242	0.251	0.251	0.647	0.647	-1.982	1.982
CTRL6	1064598.032	839645.704	1820.271	1819.303	1819.302	1819.664	1822.660	0.968	0.968	0.969	0.969	0.607	0.607	-2.389	2.389
CTRL7	1064589.123	839873.554	1871.636	1871.732	1871.736	1870.504	1871.553	-0.096	0.096	-0.100	0.100	1.132	1.132	0.083	0.083
CTRL8	1064513.791	839758.638	1860.973	1860.773	1860.778	1861.421	1863.415	0.200	0.200	0.195	0.195	-0.448	0.448	-2.442	2.442
CTRL9	1064507.689	839815.690	1882.067	1881.913	1881.903	1883.378	1883.314	0.154	0.154	0.164	0.164	-1.311	1.311	-1.247	1.247
CTRL10	1064451.462	839788.050	1869.692	1869.635	1869.633	1869.649	1872.339	0.057	0.057	0.059	0.059	0.043	0.043	-2.647	2.647
					AVERAGE ERROR			0.148	0.247	0.140	0.258	0.332	0.777	-1.828	1.844

Source: *own elaboration.*

3.3. Surface Comparison

After comparing the accuracy of individual points using the four methods under study, we compared the surfaces generated with the information collected from each. To do this, we loaded the four surfaces into the same workspace using AutoCAD Civil 3D software, as depicted in Figure 8.

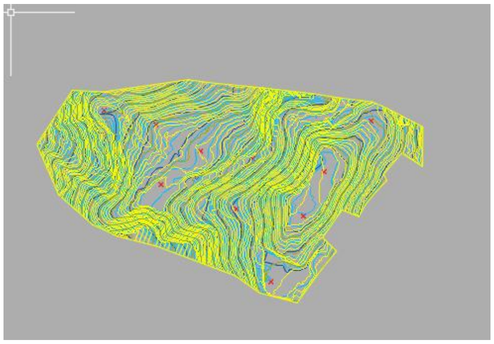

Fig. 8. *Comparison of surfaces in AutoCAD Civil 3D.*
Source: *own elaboration.*

First, we visually inspected the contour line sets obtained from each survey method. Next, we calculated the cut and fill volumes, as shown in Table 7. Finally, we divided the volume variances by the total area in the study area to determine the average height difference between the surfaces. In this comparative analysis, we used the data generated with the total station as the baseline.

Table 7: *Surfaces' Comparative report*

Volume Summary							
Name	**Type**	**Cut Factor**	**Fill Factor**	**2d Area** (square meters)	**Cut** (cubic meters)	**Fill** (cubic meters)	**Net** (cubic meters)
Comparison TE GNSS	full	1.000	1.000	39905.47	145.03	315.37	170.34<Filling>
Comparison TE-UAV PCA	full	1.000	1.000	39907.76	53760.32	10320.38	43439.94<Cut>
Comparrison TE-UAV NO PCA	full	1.000	1.000	39874.58	3069.77	74868.57	71798.80<Filling>

Source: *own elaboration.*

3.3.1. Total Station & GNSS Receivers

Upon visually inspecting the two surfaces outlined using data obtained with the total station and GNSS antennas, we observed that the contour lines had a similar shape and were only millimeters apart. The two surfaces overlapped or slightly intersected in some areas, showing significant similarity. The calculation of volumes confirmed the similarity between the two surfaces, as we obtained an average difference of -0.004 meters when dividing the total cut and fill by the total area of the study zone.

3.3.2. Total Station & UAV With Control Points

Even though the two surfaces differ, they are generally similar, as both surfaces describe the morphology of the area we studied. The areas with tall grass showed more discrepancies, as the image processing software could not accurately determine the terrain's actual height in those spots. After analyzing the images using the software, we found an average height difference of 1.089 meters, which aligns with our previous findings.

3.3.3. Total station & UAV Without Control Points

In this case, we observed similar but sharper patterns compared to the previous case, as the differences are even more prominent. When we analyzed the data using AutoCAD Civil 3D software, we found the most significant variation between the methods studied in this research, with an average of 1,799 meters. These noticeable differences demonstrate the importance of setting aerial control points to support data collection and processing tasks.

4. CHRONOLOGICAL ANALYSIS

In this study, we distinguished between field and office work when analyzing the performance of each survey method. When examining the time performance of the total station, we took into account the time needed for polygonal creation, setting the station in terrain, and locating the prism. For the GNSS antennas, we considered the time the Rover antenna needed to fix the point to be referenced. Additionally, we factored in the team's displacement time from the office to the field, the lunch hour (included in the "rest time" item if the field time exceeded 4 hours), hydration breaks (included in the "radiation of all deltas" for the total station, and "point acquisition" for the GNSS antennas). It is important to note that since the total station survey required two days of fieldwork, we doubled the assigned time for the "travel" section.

In our assessment of the total station's performance, we factored in the time it took to georeference two base points using GNSS antennas, as none were available in the area. We included this item because failing to reference these points would have caused a rotation in the entire survey and resulted in lower absolute accuracy. We also considered this factor when analyzing the time performance of the UAV with control points, as georeferencing air control points using this method has become the standard.

We included an "initial inspection" item in all the methods we examined. This task involved surveying the terrain before beginning the survey process to identify the optimal locations for setting the base and delta survey points, recognizing any potentially hazardous areas, and determining the best location for launching the drone. Table 8 provides a comparative summary of the fieldwork time required for the four methods we analyzed.

Table 8: *Comparison of time required to carry out field work*

T total(min)		TOTAL STATION	t (min)	GNSS ANTENNAS	t (min)	UAV WITH PCA	t (min)	UAV NO PCA	t (min)
30	FIELD WORK	TRAVEL TO THE SITE (2 DAYS)	120	TRAVEL TO THE SITE	60	TRAVEL TO THE SITE	60	TRAVEL TO THE SITE	60
60									
90				INITIAL INSPECTION	30	INITIAL INSPECTION	30	INITIAL INSPECTION	30
120				BASE POINT WITH GNSS	30	BASE POINT WITH GNSS	30	UAV FLIGHT AND READINESS PLAN	60
150		INITIAL INSPECTION	30	COLLECT POINTS	240	COLLECT AERIAL CONTROL POINTS (PCA)	60		
180		BASE POINT WITH GNSS	30					FLIGHT	30
210		POLYGONAL	120			UAV FLIGHT AND READINESS PLAN	60	BACK TO THE OFFICE	60
240									
270						REST TIME	60		
300									
330		DELTAS ORIENTATION	90			FLIGHT	30		
360						BACK TO THE OFFICE	60		
390				REST TIME	60				
420		REST TIME	60						
450				BACK TO THE OFFICE	60				
480		RADIATION OF ALL DELTAS	300						
510									
540									
570									
600									
630									
660									
690									
720									
750									
780		BACK TO THE OFFICE (2 DIAS)	120						
810									
840									
870									
TOTAL FIELD TIME (min)		**870**		**480**		**390**		**240**	
TOTAL FIELD TIME (hour)		**14.5**		**8**		**6.5**		**4**	

Source: *own elaboration.*

We needed 14.5 hours to capture the necessary data using the total station, equivalent to 2 days of fieldwork based on an 8-hour work day. Most of the time spent utilizing this method corresponds to the time needed to change the base point (in this case, we had to set 5 different deltas). On the other hand, to carry out measurements with GNSS antennas, all the points had to be taken directly in the field. The total time required for this task with the GNSS equipment was 8 hours, equivalent to one day of work. As expected, data collection with UAVs required less time in the field. In the case of UAV measurements without aerial control points, the required time was 4 hours, and with aerial control points, it was 6.5 hours. However, it is important to note that while data collection without aerial control points is the fastest survey method, it has lower relative and absolute accuracy compared to the other methods analyzed.

The office work involved creating a contour line map for the area under study. During this phase, the photogrammetric processing step took the most time, with a total of 11.5 hours for UAVs with aerial control points and 11 hours for UAVs without aerial control points. However, it is essential to remember that most photogrammetric processes are automatic, and the speed with which they are completed depends mainly on the power of the computer equipment. For the creation of the area map, the only processes that needed the team's attention during its development were the rectification with control points, the manual classification of points, and the processes common to all, such as the item of surface creation and plans elaboration. Meanwhile, the difference between the times needed to complete the office work with total station and GNSS antennas lies in the need to time we assigned to calculate and adjust the polygonal. In this last case, the technology that required less time was the GNSS antennas. Table 9 details the office work time associated with each method analyzed.

Table 9: *Comparison of the time required to perform office tasks*

T total(min)		TOTAL STATION	t (min)	GNSS ANTENNAS	t (min)	UAV WITH PCA	t (min)	UAV NO PCA	t (min)
30	OFFICE WORK	CALCULATION AND ADJUSTMENT OF THE POLYGONAL	120	POINT LOADING AND FEATURE ASSIGNMENT	60	ORIENTING PHOTOGRAPHS CREATING A SCATTERED POINT CLOUD	30	ORIENTING PHOTOGRAPHS CREATING A SCATTERED POINT CLOUD	30
60						PCA RECTIFICATION	30	DENSE POINT CLOUD CREATION	120
90				SURFACE CREATION WITH FEATURES, PROPERTIES, LABELING, TRIANGULATIONS AND CLEANING OF OUT-OF-RANGE OR CROSSED CURVES	180	DENSE POINT CLOUD CREATION	120		
120									
150		POINT LOADING AND FEATURE ASSIGNMENT	60						
180								COLORING POINTS	30
210		SURFACE CREATION WITH FEATURES, PROPERTIES, LABELING, TRIANGULATIONS AND CLEANING OF OUT-OF-RANGE OR CROSSED CURVES	180			COLORING POINTS	30	AUTOMATIC CLASSIFICATION OF POINTS	30
240						AUTOMATIC CLASSIFICATION OF POINTS	30	MANUAL RECTIFICATION	30
270				ELABORATION AND EXPORTATION OF PLANS	90	MANUAL RECTIFICATION	30	MESH AND MASKS	120
300						MESH AND MASKS	120		
330									
360									
390		ELABORATION AND EXPORTATION OF PLANS	90					TEXTURING, TESSERAE AND MDE	30
420						TEXTURING, TESSERAE AND MDE	30	ORTHOMOSAIC AND CONTOUR LINES	30
450						ORTHOMOSAIC AND CONTOUR LINES	30	SURFACE CREATION WITH FEATURES, PROPERTIES, LABELING, TRIANGULATIONS AND CLEANING OF OUT-OF-RANGE OR CROSSED CURVES	150
480						SURFACE CREATION WITH FEATURES, PROPERTIES, LABELING, TRIANGULATIONS AND CLEANING OF OUT-OF-RANGE OR CROSSED CURVES	150		
510									
540									
570									
600								ELABORATION AND EXPORTATION OF PLANS	90
630						ELABORATION AND EXPORTATION OF PLANS	90		
660									

TOTAL OFFICE (min)	**450**		**330**		**690**		**660**	
TOTAL OFFICE (hour)	**7.5**		**5.5**		**11.5**		**11**	

Source*: own elaboration.*

The total working time required to complete the work with the total station was 22 hours, that required for the GNSS antennas was 13.5 hours, that for the UAV with air control points was 18 hours, and as expected, the UAV execution without air control points required the least amount of time, taking a total of 15 hours.

Table 10: *Total Execution Times*

	TOTAL STATION		**GNSS ANTENNAS t (min)**		**UAV WITH PCA t (min)**		**UAV NO PCA t (min)**
TOTAL FIELD TIME (min)	**870**		**480**		**390**		**240**
TOTAL FIELD TIME (hour)	**870**		**480**		**390**		**240**
TOTAL OFFICE TIME (min)	**450**		**330**		**690**		**660**
TOTAL OFFICE TIME (hora)	**7.5**		**5.5**		**11.5**		**11**
TOTAL TIME (min)	**1320**		**810**		**1080**		**900**
TOTAL TIME (hours)	**22**		**13.5**		**18**		**15**

Source: *own elaboration.*

It is important to note the potential for significant time savings in UAV data processing by using alternative methods. For instance, skipping the creation of the texture net and tessellation model and going directly from point classification to the digital elevation model can reduce processing time. Future research should consider this variable and compare surfaces processed with and without these steps. If these processes were omitted, the total execution time for UAV with aerial control points would be 13.5 hours, while UAV without aerial control points would require only 10.5 hours.

5. CONCLUSIONS

The total station is still ideal equipment if a survey assignment requires the highest possible accuracy. However, this accuracy is achieved at the cost of a considerable increase in execution time, as it may require more than twice the time other alternatives we analyzed in this study need.

GNSS antennas with the RTK method are a strong contender in terms of accuracy and execution time, offering a viable alternative to the total station. They are not only suitable for preliminary surveys but also for quantities control or surveys that demand high geomorphological representation accuracy. This technology delivers outstanding performance, largely due to its excellent satellite reception and seamless communication between Base and Rover during data collection. However, it is important to note that the absence of either of these conditions could lead to significant data degradation, potentially resulting in errors in the vertical and horizontal above the meter., which could make the method unsuitable for high-precision altimetric surveys.

Finally, we found that the accuracy of processing images taken with non-specialized UAVs is affected in areas with varying heights and constant changes in slope and morphology. Due to these limitations, we recommend carrying out a proper densification and setting multiple control points in the area, especially in very irregular locations, to enhance absolute and relative accuracy [13]. Despite these limitations, using UAVs is still an excellent option for representing geomorphological features in an area when centimeter-level accuracy is not necessary. Additionally, this technology offers the benefit of reducing fieldwork time, accessing inaccessible or hazardous areas, and producing products like orthomosaics, which provide more detailed information than a digital elevation model or contour lines.

It is important to note that the UAV's performance on the ground was affected by various external conditions. First, the UAV experienced unfavorable weather conditions, including strong winds with gusts exceeding 40 km/h during the flights. These conditions made it challenging to execute the flight plan for photogrammetric purposes, as stability and constant speed are required. The high yaw error in the photogrammetric processing report confirmed this issue. Second, there was a significant decrease in GPS signal quality at different points during the flight plans. Finally, in this study, only 5 aerial control points were set, which later proved insufficient. All these conditions help to explain why previous studies or projects by the same authors had better results, especially in flat areas or planimetric tasks where accuracy below 5 centimeters was achieved.

In sum, we cannot definitively state which technology or equipment is better. It depends on the work's objectives, the required precision, geomorphological and climatic conditions, and available time and resources. Additionally, for certain projects, using multiple technologies may be necessary due to the needs of the products to be delivered, climatic conditions, or the geomorphology of the terrain to be studied.

ACKNOWLEDGMENTS

This paper is derived from the research initiation project PIC-DIS-3879, funded by the Vicerrectoría de Investigaciones of the Universidad Militar Nueva Granada. Additionally, the authors give credit to the Universidad Militar Nueva Granada for the support received in the development of this research and in the preparation of this scientific document.

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