Journal of Geomatics and Environmental Research, Vol. 2, No. 1, December 2019
ISSN 2682-681X (Paper) ISSN 2705-4241 (Online) |
https://unilorinjoger.com/

TOTAL STATION AND DIFFERENTIAL GLOBAL POSITIONING SYSTEM: A COMPARATIVE SUITABILITY FOR CONTROL EXTENSION

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ABSTRACT

In control extension or any other surveying projects, more sophisticated instruments such as Total Station (TS) and Differential Global Positioning System (DGPS) are employed to improve the efficiency and accuracy of the work well as to save time and energy (Berg, 1996)). Accuracy, precision and time are factors most importantly to be given consideration in any surveying project and these factors depends enormously on the instrument used and the procedure adhered in observation. To evaluate and compare the accuracy and precision achieved with the range of spent when using the two instruments (Total Station and DGPS), reference network is established and evaluated. The reconnaissance survey was carried out followed by the data acquisition where the observations were made on reference station network using total station and DGPS respectively (Borgelt 1996). The acquired results were adjusted using least square adjustment. The results obtained were compared using RMS and Standard Deviation. Finally, it revealed that DGPS is more precise than total station and subsequently more accurate if a reasonable time of at least 30 minutes intervals is used in observation for each station.

Keywords: Differential GPS, RMS, Standard Deviation, Total Station.

1.0 INTRODUCTION

In surveying, specifically in the area of control extension, deformation monitoring and engineering projects, sophisticated instruments such as Total Station (TS), Laser Scanner and Differential Global Positioning System (DGPS) are employed to improve the efficiency and accuracy of the projects (Borgelt et al, 1996). Different surveying instruments are used in the history of surveying to collect data from field measurements for various applications with different accuracy capabilities and requirements. Thus the knowledge of these factors on any instrument is important in designing any surveying project. The required accuracy depends on the needed deliverable output (Clark, 1998).

The total station is a surveying instrument that combines the angle measuring capabilities of theodolite with an electronic distance measurement (EDM) to determine horizontal angle, vertical angle and slope distance between particular points. Or more precisely total station is version of EDM that used infra-red signals to measure the time taken for wave to reached the reflector and return to the instrument and the distance is calculated

ISSN 2682-681X (Paper) ISSN 2705-4241 (Online) |
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using the known speed wavelength through air (Dean et al. 1982). Hence the coordinates (X, Y, and Z or easting, northing and elevation) of surveyed points relative to the total station position are calculated using trigonometry and triangulation. To determine an absolute location a Total Station requires line of sight of the observation points and can be set up over a known point or with line of sight to 2 or more points with known location, called Resection (Free Stationing) and it attain accuracy of about 5mm in 4km (Dean et al. 1982). Differential Global Positioning System (DGPS) on the other hand is a navigational system using the GPS system of satellites that circle the earth plus ground stations with limited signal range. It was created by US military and became open for use by civilian from 1984 in Standard Position Service (SPS) code (Delgado, 1997). Now GPS has high profile in surveying due to its lowered cost of units and relative integration. GPS works on a system where satellite (24 satellite) in sky transmit signals to the receivers on the ground and by using the principle of trilateration, position and time information can be calculated. DGPS was introduced to correct the effect of selective availability where two receivers are used with one on base and a mobile unit data (Delgado, 1997). DGPS uses second unit close to the first and positioned on survey station, providing better accuracy of 1 –10mm (Dean et al. 1992).

Accuracy of surveying techniques using instruments such as GPS and Total Station are dependent on a number of parameters that limit their measurement quality (Clark, 1998). For instance; the multipath, the inherent satellite signal accuracy, signal transmission delay, receiver hardware and software limitations, satellite signal obstruction are some of the problems associated with GPS measurements (Solomon, 2014). On the other hand, limitations stemming from Total Station are; computed coordinates are in local or target coordinate system and reference surface for measuring height is geoid (Solomon, 2014).

Because of earth's curvature,measurementcanalso thebeaffectedaccuracyby distance (accuracy decreases with increasing the distance). Moreover surveying with a total station, unlike GPS, is not disadvantaged by overhead obstructions but, it is restricted to measurements between inter-visible points (Solomon, 2014). Often control points are located distant to the survey area, and traversing with a total station to propagate the control is a time consuming task. However, GPS can measure points without any line of sight. Since total stations work on the principle of signal reflection, line of sight must be there between total station and prism reflector. However, GPS cannot be used in dense areas, high rise buildings because of satellite signal interference. Therefore, each instrument has its own advantages and disadvantages. In practice the accuracy of surveying measurements can be improved almost indefinitely with increased cost (time, effort and money) (Chekole, 2014).

In this paper the working efficiency, accuracy and precision of Total Station and DGPS will be studied to determine their suitability for control extension. To achieve this aim the paper intended to establish and evaluate a précised reference network which can be served as the bench mark for the comparison between the two instruments. Thereafter to evaluate

ISSN 2682-681X (Paper) ISSN 2705-4241 (Online) |
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the accuracy and precision networks of Total Station and DGPS data obtained from this reference network and finally to compare results based on RMS and standard deviation analysis. Most significantly this paper can be used as a bench mark for further studies for those who are interested in comparative study of surveying instruments. Also the study can help users to choose appropriate instruments for a given task. Moreover, since coordinates of the reference points are determined with high precision, it can be serves as a reference values for other users.

1.1 Study Area

The study area is Modibbo Adama University of Technology Yola located at Sangere in Girei Local Government Area, along Jimeta- Mubi road, Adamawa State, Nigeria. The study area lies between latitude 9 ^{0}20’ 30”N21’to05”N9 ^{0} and29’longitude48”Eto 12 ^{0} 30’ 25”E. it has ^{2} a spatial extent).Thesummerofis about much rainier than the winter and therefore classified as KoppenGaiger climate with average annual temperature of 27.and5˚C940mm of precipitation falls annually. Below is study area;

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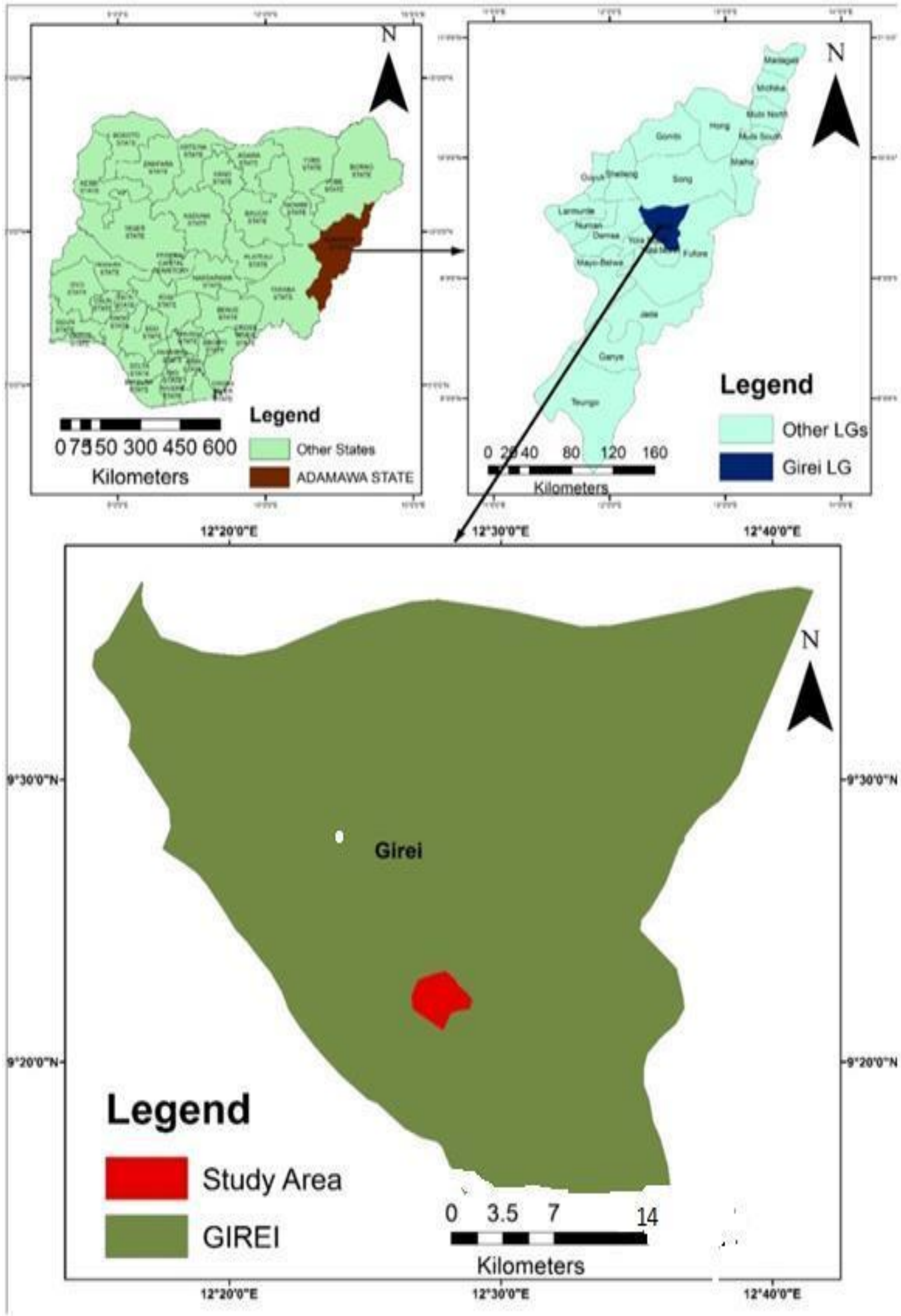

Figure1: Map of Adamawa state showing the study area.

2.0 LITERATURE REVIEW

The fundamental ideal underlying is this paper is based on the concept of statistical root mean square and standard deviation error techniques to assess the positional accuracies and precisions of two sophisticated surveying instruments (Total Station and Differential GPS). The establishment of two-or-three-dimensional control networks is the most fundamental operation in the surveying of an area of large or small extent (Schofield, 2001). Naesset et al (2001) used a 20-channel, dual-frequency receiver observing dual-frequency pseudo range and carrier phase of both GPS and GLONASS, to determine the positional accuracy of 29 points under tree canopies. Yoshimura et al, 2003 checked the performance of GPS in forest area to turn-off SA and field test on horizontal and vertical position error of GPS positioning at different point in forested area. He finally portrayed that precision and accuracy errors of autonomous GPS in plantation forest area were 2.16m –6.79m and 3.26m

–6.19m for horizontal and vertical direction respectively. Kumar et al (2013) work on Horizontal Accuracy Assessment of Differential-GPS Survey summarize that 25-minute observation time is sufficient as the accuracy in horizontal measurement for 25-minute observation Standard deviation and Standard Error is 0.013 meter and 0.003 meter respectively. Thus accuracy of DGPS survey is dependent upon the observation time. It is also affected by the PDOP (Position Dilution of Precision). Fregonese, et al (2007) studied that the feasibility of monitoring deformations of large concrete dams using terrestrial laser scanning. For this purpose, a test field has been established on the specific dam. First the author established a geodetic network as a reference by Leica TS, and then, using a number of targets on the dam, measurements were taken with a total station and a laser scanner. The reference network was determined with 2 mm horizontal and 3 mm vertical coordinate precision. Targets, mounted on the dam, were measured precisely with a total station, and 3 mm for the horizontal and 4.5 mm for the vertical coordinate accuracy (RMS) has been achieved. On the other hand, using a laser scanner (HDS 300), 4 mm for the horizontal and 8 mm for the vertical coordinate accuracy (RMS) was achieved. In this paper a comparative study in accuracy and precision between total station and DGPS is

3.0 METHODOLOGY

Three (3) existing control points established with high accuracy (2nd order) were used to establish six (6) reference networks using RTK. To determine the network with high precision one hour time elapse was used for each station and the coordinates were processed using free adjustment. On the established reference control network; observation were carried out five times on each control point using Total Station. Observations were taken in two faces with two rounds to minimize errors such as line of sight errors, tilting axis errors and vertical index errors. Data from the Total Station were processed in Geo Cal. On the same reference network observation were carried out using Pro –mark 3 DGPS with 30minute time elapse. The data were processed using GNSS solution software. Then, precision of the network was

ISSN 2682-681X (Paper) ISSN 2705-4241 (Online) |
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obtained from network adjustment. Adjustments of the networks were performed in Microsoft Excel software using the method of least square adjustment which stated that;

Where: V = is the residual vector

P = is the weight matrix of the observations.

The above estimate is referred to as Least Square Estimate. The process of obtaining the least square estimate is known as the Least Square Adjustment in surveying.

Functional Model Stochastic Model

Condition Equation; in this method, the observations are expressed in a functional relationship that ignores the parameters (i.e. the unknown to be found).

The model is given by: $f\,(l) = 0$ (1)

3.1 Evaluation of Accuracy and Precision

To evaluate the accuracy and precision of the measurement, root mean square (RMS) and standard deviation (SD) of the individual measurements were computed. RMS was computed using the following formula:

(2)

Where = is the established value,

= is individual measurement and

= is the number of measurements

On the other hand; Standard Deviation is given by;

(3)

Where; is the true or established values obtained by control extension from existing control points (table 2).

= is individual measurement.

= is mean value of the measurements.

n = is number of measurements.

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4.0 PRESENTATION OF RESULTS

Table 1 Processed and Adjusted Coordinates of the Three Control Points Using DGPS

Points	Easting (E)	Northing(N)
CP1	1383234.632	1058796.444
CP2	13833152.875	1059051.238
CP3	1382970.312	1058831.956

Table 2 Adjusted Coordinates of the Reference Network of the Control Points

Points	Easting (E)	Northing(N)	σE	σN
1	1383278.798	1058805.937	0.002	0.003
2	1383378.932	1058843.435	0.006	0.005
3	1383339.099	1058949.031	0.006	0.004
4	1383290.582	1059061.324	0.005	0.006
5	1383189.506	1059030.023	0.006	0.006
6	1383230.862	1058917.723	0.007	0.006

Table 3: Adjusted Coordinates of the Total Station with it RMSE and Standard Deviations (σ)

Points	Easting (m)	Northing(m)	X(m)	Y(m)	X(m)	Y(m)
	Total Station Mean		RMS		St.D (σ)	
1	1383278.798	1058805.937	0.000	0.000	0.008	0.006
2	1383378.938	1058843.439	0.006	0.004	0.007	0.007
3	1383339.104	1058949.034	0.005	0.003	0.006	0.007
4	1383290.589	1059061.330	0.007	0.006	0.009	0.009
5	1383189.510	1059030.030	0.004	0.007	0.008	0.008
6	1383230.864	1058917.725	0.002	0.002	0.007	0.006

ISSN 2682-681X (Paper) ISSN 2705-4241 (Online) |
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Table 4: Adjusted DGPS coordinates and their computed RMSE and Standard Deviations (σ

Points	Easting (E)	Northing(N)	X	Y	X	Y
	DGPS Mean		RMS		St.D (σ)	
1	1383278.773	1058805.924	0.025	0.013	0.019	0.0017
2	1383378.904	1058843.422	0.028	0.013	0.017	0.018
3	1383339.113	1058949.046	0.014	0.015	0.015	0.016
4	1383290.596	1059061.313	0.014	0.011	0.018	0.016
5	1383189.515	1059030.032	0.009	0.009	0.015	0.019
6	1383230.844	1058917.699	0.018	0.024	0.018	0.016

Table 5 Difference in coordinate between Total Station and DGPS

Points	Total Station mean		DGPS mean		Total Station –DGPS	
	Easting (E)	Northing(N)	Easting (E)	Northing(N)	ΔN	ΔE
1	1383278.798	1058805.937	1383278.773	1058805.924	0.025	0.013
2	1383378.938	1058843.439	1383378.904	1058843.422	0.034	0.017
3	1383339.104	1058949.034	1383339.113	1058949.046	-0.009	-0.012
4	1383290.589	1059061.33	1383290.596	1059061.313	-0.007	0.017
5	1383189.51	1059030.03	1383189.515	1059030.032	-0.005	-0.002
6	1383230.864	1058917.725	1383230.844	1058917.699	0.02	0.026

Table 6 Difference in Standard Deviation between Total Station and DGPS

Points	Total Station		DGPS		Difference	
	σE	σN	σE	σN	∂σE	∂σN
1	0.008	0.006	0.019	0.017	-0.011	-0.011
2	0.007	0.007	0.017	0.018	-0.01	-0.011
3	0.006	0.007	0.015	0.016	-0.009	-0.009
4	0.009	0.009	0.018	0.016	-0.009	-0.007
5	0.008	0.008	0.015	0.019	-0.007	-0.011
6	0.007	0.006	0.018	0.016	-0.011	-0.01

5.0 DISCUSSION OF RESULTS

The coordinates of the 3 control reference network are presented in table 1. Table 2 represented the coordinate of reference network and their RMS error. The errors ranged from 2mm to 7mm which indicted that high accuracy had attained for the reference points and can serve as bases for the comparison. To determine the precision of the repeated measurement (Total Station and DGPS) of the reference network, standard deviation formula Eq. (3) has

ISSN 2682-681X (Paper) ISSN 2705-4241 (Online) |
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been used. Then, Root Mean Squares of the DGPS and Total Station measurements were computed using Eq. (2) and these evaluated how much the measurements were close to the established value. Adjusted coordinates of the Total Station with it RMSE and Standard Deviationswasshown (σ)intable 3. Adjusted DGPS coordinates, RMSE and Standard Deviationswereshown in(σ)table 4. Table 5 represent the difference in coordinate between Total Station and DGPS and finally Table 6 represented the difference in Standard Deviation between Total Station and DGPS. As the result shows in Tables 6, the standard deviations are less than 8 mm which indicated that the repeated measurements were quite close to each other (Jonsson et al. 2003). He stated that the standard deviations for the horizontal and vertical coordinate are 9 mm and 2 cm respectively. So, by comparing the author's resu with this paper`s result, the precisions of the coordinates are very high. The paper results are quite reasonable considering the errors attributed from satellite blocking, centering error and so on.

6.0 CONCLUSION

This focus of this paper was to evaluate and compare accuracy and precision between TS and DGPS. To accomplish the objectives of the paper, three major tasks have been performed. 1. A network of 6 control points was established with high precision (lmm) with RTK. Control extension was performed using Total Station and DGPS on the same network to compare the results. In each case the data were processed and adjusted. The RMS and Standard Deviation of each network were calculated and their differences were determined. Finally, comparison has been made between the established coordinates of DGPS and those measured by the total station based on accuracy and precision.

Based on the results obtained, precision of the reference network determined with 1 mm standard deviation. The accuracy of the RTK measurements on the network, which is expressed by RMS, is less than 9 mm. Precision of the TS measurement on the other hand has been determined with maximum standard deviation of 8 mm. On the same points, coordinates obtained from the DGPS measurement has been determined with maximum standard deviation of 9 mm. Then the accuracy of the Total Station measurements was determined with maximum RMS of 4 mm. Based on this quality control measurement, more than 95% of the total result has achieved the requirement. This can be interpreted as value lied within the allowable limit (interval limit) and considered as accepted values. Thus, it can be concluded that there were no gross errors in the measurement; because the measurements were made precisely and accurately. For instance: when measuring using total station, two face measurements was taken to eliminate some errors such as collimation axis errors, tilting axis errors, etc. When using GPS, small tripod was used to erect the rover vertical. Subsequently it's also observationdeterminedwithDGPSis morethatefficient as portrayed in the results. Moreover it would be recommended that Total station should be calibrated at

ISSN 2682-681X (Paper) ISSN 2705-4241 (Online) |
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some regular intervals. It can be achieved better accuracy by calibrating those instruments before the measurement campaign. Also, it was very difficult to manage the field measurement alone, specially establishing the reference network has been a big problem. There will be a possibility of occurring gross errors and therefore, working in group is recommended.

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