Evaluate Accuracy Between RTK GPS and Total Station in Adjustment Closed Traverse Using the Least Square Method.

Mohamed Faraj EL Megrahi
Lecturer, Higher Institute for Building Trades and Construction, Benghazi, Libya.
mohamedbaleid79@gmail.com

Abstract

The necessity to implement high precision engineering projects requires the use of precise surveying equipment. The most surveyors currently used high precision instruments like of the Global Position System GPS and the Total Station TS. The aim of this research is evaluation the precision assessment between Real time kinematic RTK GPS and TS via measuring of close traverse by applying the least square method by using Map check tool in Civil3D program. Furthermore, two-dimensional 6 points closed traverse was recognized and include at the Higher Instate for Building Treads and Construction campus, Benghazi city-Libya. This study the accuracy difference between the instruments is through adjustment of 6 points closed traverse. Root mean square error RMSE and standard deviation SD were used to evaluate the accuracy and precision of the measurement by RKT and TS. The accuracy of the RTK device was better than the TS device with absolute errors 0.057 m and RMSE for East 0.018 m, and North 0.021 m besides SD= 2.53. RKT can measure points without any line of sight requirement. However, RTK based survey is not only practical and efficient but also time-saving than TS and more efficient in human resources. Therefore, investment in RTK GPS system might be worthwhile.

Keywords: RKT GPS, Total station, closed traverse, least square, Civil3D software.

1. INTRODUCTION

Control measurement is that a part of measurement during which high preciseness instruments and techniques utilized to find points for succeeding surveying operations. Therefore, a correct estimate is key for the success of any engineering project. Near an inaccurate survey results, that does not represent the construction area will make a biased or an inaccurate investment. Specifically within the space of engineering comes, additional subtle instruments (total station, and GPS) to boost the potency and accuracy. The accuracy of measurements will be improved virtually indefinitely with enhanced value (time, effort and money). The Global Positioning System - Real Time Kinematic GPS-RTK system is taken into account the foremost helpful system for geographic surveys between satellite survey technologies. RTK primarily based surveys are not only sensible and quick however additionally yield additional correct geographic maps for style purpose [1]. In several construction monitoring processes, observations obtained by Total Station TS. What is more, it's expressed that TS could be an additional appropriate technique to conduct, geographic measurement than employing a surveying instrument for personal firms inside Libya, whether this is often undertaken by the owner, contractor or adviser. Not only will this instrument give smart accuracy however provides automatic computation for the info [1]. A close-loop traverse could be a closed form encompass each measurement angles and distance, the pure mathematics summation of departure and latitude departure are zero. A traverse has 3 olden adjustment ways. The primary technique is the Compass Rule adjustment. The Compass Rule adjustment assumes that each the discovered angles, and also distances, square measure in error. Another technique, the Transit Rule adjustment, assumes that the best quantity of error is within the distances of the traverse. The Crandall Rule adjustment assumes there square measure errors in distance measurements associate distributes an angular error throughout the traverse. As a final technique, the least square adjustment on the traverse knowledge supported a traverse has redundant observations (forward and backwards) on the traverse course. A method of least squares adjustment tries to seek out the amount of error for all discovered traverse points. So direct changes and indirect change measurements computation of parameters procedure was used Civil 3D engineering programming is a straightforward to use application that adjusts second and 3D survey networks use method of least squares techniques. Many studies are conducted within the space of analysis the RTK and TS in land measurement for construction practices, such as:

Lao-Sheng Lin,2004, in his study, the campus of National Cheng-Chi University-China was selected as a test region to test the performances of RTK and TS system on land use change data collection. His study results indicate that, the horizontal accuracies of RTK and TS system were14 mm and 163mm respectively, and the time required for one point determination using RTK or TS system were about 15 seconds and 240 seconds respectively. According to that can be concluded the RTK is one of the best choices to collect coordinates of the land use change region[2].

El-Mowafy ,2012, in his study, investigated the RTK achievable accuracy and repeatability under different satellite configurations and site conditions in an urban environment used control station in the city of Al-Ain, in the United Arab Emirates. His study results show that the RTK is more stable for the horizontal coordinates than the height, and data latency generates errors of a few millimeters. Furthermore the RMS of the differences, were less than 3 cm in general. However, The RTK was also capable, at all times, of achieving cm level accuracy throughout the changing geometry of satellites, within the urban environment, when sufficient number of satellites was observed. The RTK system is therefore very useful, particularly for ambiguity resolution in the urban environment [3].

Chekole, 2014, The objective of his study is to evaluate and compare precision, accuracy and time expenditure of TS, RKT and terrestrial laser scaner (TLS). His study area was the parking lot close to L building, KTH campus, Stockholm, Sweden. According to his study result obtained, the coordinates of the six target points measured with TS were determined with a standard deviation of 8 mm for horizontal and 4 mm for vertical coordinates. Also When using RTK method on the same reference network points, 9 mm in horizontal and 1.5 cm accuracy in vertical coordinates has been achieved. As well using TLS for the same target points, 2mm accuracy has been achieved for both horizontal and vertical coordinates. With regard to time expenditure, it is proved that total station consumed more time than the other two methods (RTK and TLS)[4].

Elhassan,2017, his study objective is to evaluate the horizontal positioning accuracy of a RTK technique using reference network of eleven control points within King Saud university compass area.

His study Results show that a horizontal positional RMSE of 11 mm can be obtained using the RTK. And his conclusion is RTK can be used for many tasks within different applications and documentation of cultural heritage with high accuracy level (up to 11 mm in horizontal positioning)[5].

Mohammad Idris,2019, Accuracies of GPS-RTK system and TS were investigated in his research at 5.90 ha salt ponds area near Gresik city, Indonesia. Based on his research results, it had been revealed that RTK based survey is not only practical and efficient but also yields acceptable accurate topographic maps for medium accuracy construction purposes[1]. Ameen, et al,2019, in their study they used Map check tool in Civil3D program for evaluation the accuracy assessment between RTK and TS by measuring of close traverse of eleven points located at the middle south of Kirkuk City - Iraq. According to their research results the northing and easting errors of RTK were 0.0098 m and 0.0126m and for TS were 0.092 m and -0.056 m respectively. The accuracy of the RTK device was better than the TS device in both angular and linear misclosure errors as well as in separation distances of linear errors[6].

The aims of this research are to evaluate the accuracies of RTK and TS in land surveying for construction practices, through measuring 6 points on form of close traverse and using Map check tool in Civil3D program. Finally evaluating the time cost of both systems.

2.METHODOLOGY AND AREA OF STUDY

1.2 Study Area:

The study area is located at the Higher Instate for Building Treads and Construction campus, Benghazi city-Libya. The study area is bounded by geographic latitudes (20^0.049 N – 20^0.053 N) and geographic longitudes (30^0.019 E – 30^0.014 E) as presents at Figure1.

2.2 Traverses Field Works:

To evaluate the accuracy of the surveyed data, primary it has been established a control point which can serve as start and closed reference for RTK and TS devices measurements. One point was measured with static Global Position System GPS (BM=R1, and T1) to serve as a reference value. A close traverse has been established with 6 points starts and end at point R1 that indicates the RKT device. Dual rate GPS and GLONASS system was used to obtain accurate result. Furthermore, the same traverse 6 points starts and end at T1 referring to TS device. The data were measured using RTK projected by UTM (Universal Transverse Mercator). As well the Zone 34 N based on ellipsoid defined of LDG2006 was the reference system of work. The topographic datasets were stored as point measurements, each one had Northing, Easting and elevation values. Table 1, illustrates the data had been collected from the field survey also points location and order presents at Figure 2. In this study, Leica TS 06plus device was used, with the distance measurement accuracy and angular measurement accuracy are 5", ±2 mm respectively. The RTK Hipper II GPS was also used and the distance accuracy of horizontal and vertical are 10 mm, 15 mm respectively.

Table 1: The Data Collected From The Field Survey

No.	E m	N m	No.	E m	N m
Start point T1, R1, (BM)	0410405.64	3542496.08	Start point T1, R1, (BM)	0410405.64	3542496.08
TS			RTK		
T2	410456.27	3542593.45	R2	410456.22	3542593.50
T3	410498.49	3542761.09	R3	410498.41	3542761.03
T4	410361.29	3542838.37	R4	410361.36	3542838.24
T5	410283.68	3542643.56	R5	410283.78	3542643.51
T6	410242.87	3542521.72	R6	410242.94	3542521.81
End pointT7	410405.62	3542495.99	End pointR7	410405.67	3542496.03

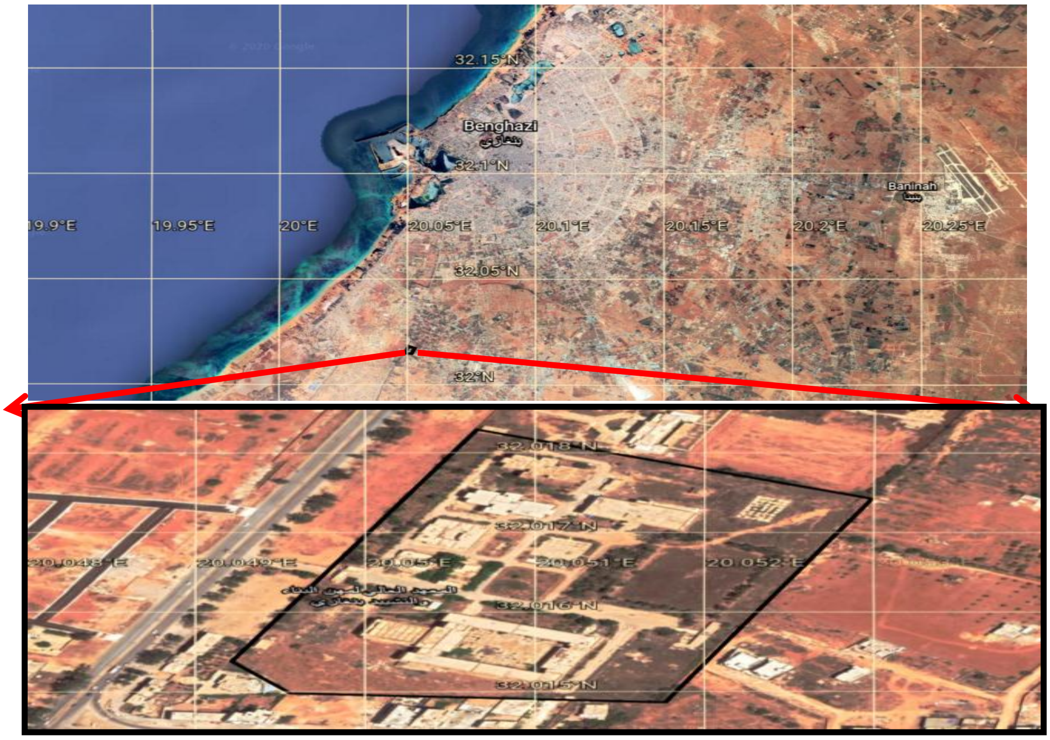

Figure. 1 The study area .

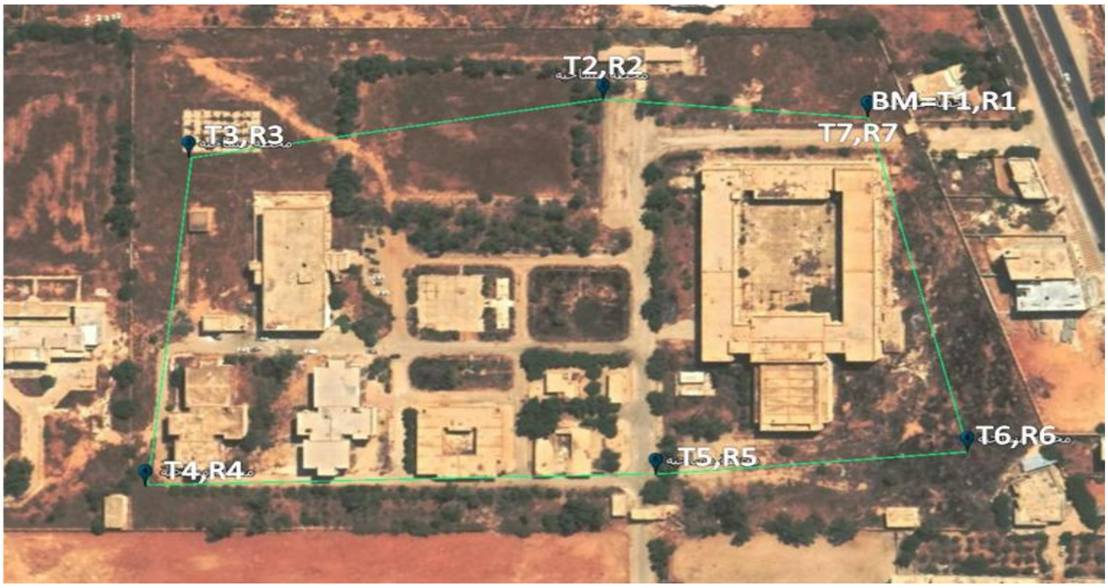

Figure. 2 Polygon Points on The Study Area

2.3 Least Squares Traverse Adjustment:
Least square adjustment is a method of estimating values from a set of observations by minimizing the sum of the squares of the differences between the observations and the values to be found. Least squares method is a classical method which defines the optimal estimate of X (unknown) by minimizing the sum of the weighted observation residuals squared [4].

$$\sum_{i=1}^{n} V_i^2 = minimum \quad (1)$$

Where:
V: residual vector .
n: number of observations.
In surveying, all equations involving observations can be reduced, by linearization where necessary to [7]:

$$AX = b + V \quad (2)$$

The coordinates are only part of the solution. An examination of the residuals will show quickly if there are any major discrepancies in the observations, residuals are[7]:

$$V = AX - b \quad (3)$$

This part of the solution is usually referred to as the normal equations, and is solved for X as [7]:

$$X = (A^T WA)^{-1} A^T Wb \quad (4)$$

Where:
X : from one observation equation.
A: matrix represents the coefficients of the terms in the X vector.
b:the vector contains all the numerical terms in the observation equation.
W: the weight matrix.
The distance between two points i and j can be related to their eastings and northings by the equation [7]:

$$l_{ij} = \left\{ \left(E_j - E_i\right)^2 + \left(N_j - N_i\right)^2 \right\}^{\frac{1}{2}} \quad (5)$$

Differentiating with respect to the parameters and change of sign and reordering the terms, the equation may be more simply expressed in matrix notation as [7]:

An angle is merely the difference of two bearings. In the current notation it is the bearing of point *k* from point *i* minus the bearing of point *j* from point *I* (Figure 3).

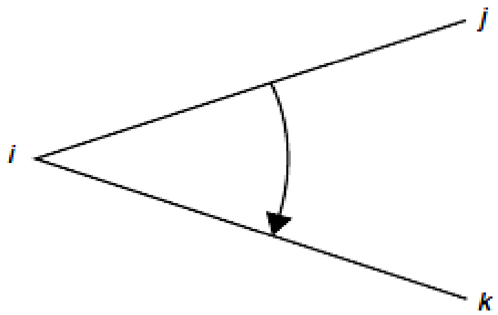

Figure 3. Angle equation.

The angle equation is[7]:

$$a_{jik} = \tan^{-1}\frac{E_k - E_i}{N_k - N_i} - \tan^{-1}\frac{E_j - E_i}{N_j - N_i} \qquad (10)$$

$$\left[\left\{\frac{\cos a_{ij}}{l_{ij}\sin 1''} - \frac{\cos a_{ik}}{l_{ik}\sin 1''}\right\}\left\{\frac{\sin a_{ik}}{l_{ik}\sin 1''} - \frac{\sin a_{ij}}{l_{ij}\sin 1''}\right\}\frac{-\cos a_{ij}}{l_{ij}\sin 1''}\;\frac{\sin a_{ij}}{l_{ij}\sin 1''}\;\frac{\cos a_{ik}}{l_{ik}\sin 1''}\;\frac{-\sin a_{ik}}{l_{ik}\sin 1''}\right]\begin{bmatrix}\delta E_i\\ \delta N_i\\ \delta E_j\\ \delta N_j\\ \delta E_K\\ \delta N_K\end{bmatrix} = \left[a_{ijK\,(o-c)}\right] \qquad (11)$$

Different surveyors may make different observations with different types of instruments. Therefore the quality of the observations will vary and for a least squares solution to be rigorous the solution must take account of this variation of quality.

If all the terms in the observation equation are divided by the assumed standard error of the observation, then the statistically expected value of the square of the residual will be 1. Where **W** is a diagonal matrix and the terms on the leading diagonal, w_{ii}, are the inverse of the respective standard errors of the observations squared and all observations are uncorrelated. The standard error of the observation and the weight matrix are [7]:

$$\sigma_b = \begin{bmatrix}\sigma_{11} & 0 & 0 & \cdots\\ 0 & \sigma_{22} & 0 & \cdots\\ 0 & 0 & \sigma_{33} & \cdots\\ \cdots & \cdots & \cdots & etc\end{bmatrix} \qquad (12)$$

$$W = \begin{bmatrix}\sigma_1^{-2} & 0 & 0 & \cdots\\ 0 & \sigma_2^{-2} & 0 & \cdots\\ 0 & 0 & \sigma_3^{-2} & \cdots\\ \cdots & \cdots & \cdots & etc\end{bmatrix} \qquad (13)$$

Where:

$\sigma_1^2 = \sigma_{11,etc}$.

σ_b : the variance-covariance matrix of the estimated observations.

σ_{11} : the variance of the first observation, and σ_{23} is the covariance between the 2nd and 3rd observations, etc.

2.4 Determination of Coordinates Using The Traverse Method:

Traverse is a method widely used in surveying and geodesy specialists for small stretching area. Basically it starts from a point with known coordinates is aimed at a different point which is unknown coordinates(Figure 4). From this point on aiming the next point traverse, linked with the above point and thus can determine the orientation of departure. Basically coordinates are determined step by step and finally closes on a known point. Coordinates transmitted by traverse should be close to those of inventory, given the precision of the old network and traverse accumulated errors. General formulas for determination are [7]:

$$E_B = E_A + D_{AB}\sin\theta_{AB} \qquad (14)$$

$N_B = N_A + D_{AB}\cos\theta_{AB}$ (15)

Between transmitted coordinates and coordinates existing difference occurs is called the traverse discrepancy in closing.

$e_x = E_B^{T_r} - E_B$ (16)

$e_y = N_B^{T_r} - N_B$ (17)

$e_t = \sqrt{e_x^2 + e_y^2}$ (18)

This error should not exceed the tolerance required T_r. Tolerance is given by the machine precision with which they work. Balancing the traverse, sometimes referred to as 'adjusting' the traverse, involves distributing ΔE and ΔN throughout the traverse in order to make it geometrically correct.

$$\Delta E = \frac{e_x}{D} \quad (19)$$

$$\Delta N = \frac{e_y}{D} \quad (20)$$

Where :

D: the sum of the lengths of the traverse .

ΔE and ΔN : the coordinate corrections**.**

After applying corrections, coordinates are transmit again and each point of the traverse will have new coordinates, corrected by traverse discrepancy in closing.

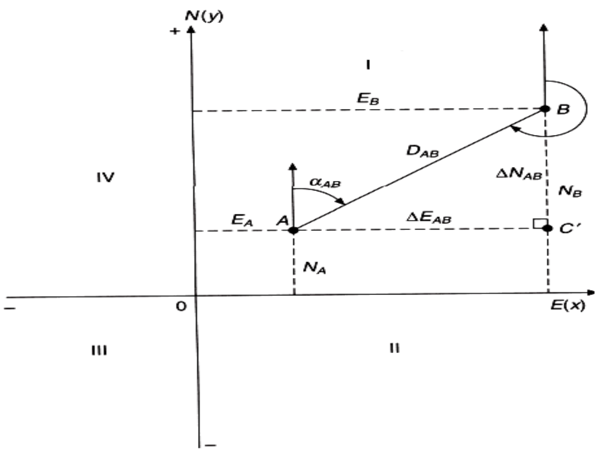

Figure 4. Plane rectangular coordinate system.

2.5 Evaluation of Accuracy and Precision

Root mean square error RMSE and standard deviation SD were used to evaluate the accuracy and precision of the measurement in this study. RMSE can be computed from the deviations between true and measured values. RMSE was computed using the following formula:

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n}(E_o - E_c)}{n}} \quad (21)$$

Where:

E_o = the observed value.

E_c = the computed value.

n= number of amusements.

Standard deviation is a measure of variations of the measurement and computed using the following formula:

$$SD = \sqrt{\frac{\sum_{i=1}^{n}(\overline{E_c} - E_c)^2}{n-1}} \quad (22)$$

$\overline{E_c}$: the average of computed value.

3. DETERMINATION OF PRECISION AND ACCURACY OF RTK AND TOTAL STATION USING LEAST SQUARE METHOD

The collected data by TS and RKT devices were transferred and interred to Civil 3D software design by text file then drawing the polygons and adjusted them by Least square method. The Mapcheck tool in Civil 3D software was applied to get the both angular and linear misclosure errors [8]. The Mapcheck tool has potential role to draw the traverses using the coordinates of points that entered to Civil3D software as well as can calculate the misscloser error for each traverse and the separation distance error of all points measured by both RKT and TS devices. Tables 2,3,and4 illustrate the summary of Civil3D software reports of the closed traverses adjustment. In addition Figures 5 and 6 present the adjusted traverses drawn by Civil3D software.

1. RESULT DISCUSSION

The comparison of point accuracy and operation time required relating to RTK and TS devices for 6 points closed traverse located on the Higher Instate for Building Treads and Construction campus was done. By noted that there is one point selected as control points (BM=R1=T1) therefore, there are 5 points are treated as check points in this study. As the reports that produced by Civil3D software as illustrated in tables 3, and 4 the error in northing and easting were -0.020m and 0.074m respectively in RKT device. While they were 0.092 m and -0.056 m respectively in TS device. In addition the absolute errors were 0.057m for RTK device and 0.077m for TS device. According to that very clearly the RTK measurements are more accurate than TS measurements. The results of previous equivalent points refer to that the separation distances of RKT instrument are more prices than the separation distances of TS instrument. Moreover, to appraise how much RTK measurements were close to the established value, the RMSE was for east 0.018 ,and north 0.021m besides SD was 2.53 both less than TS instrument results (see Table 4). Based on blunder detection report analysis produced by Civil3D software, more than 95% of the total result has achieved the requirement. This can be interpreted as values which have within the allowable limit (interval limit), considered as accepted values. it can be concluded that there were no gross errors in the measurement; because the measurements were made precisely and accurately by passing the reliability tests.

Time allocated for all steps of the measurement is presented in Table 5. Time needed to setup the tripod of TS instrument on one station was recorded and then multiplied by the number of the stations to determine the time expended on all instrument setups. Time expended for RTK was recorded as time required for the reference base and for the rover. For the reference station, time was calculated as: time required for tripod setup plus to center it which was 8 min. For the rover measurement, time has been recorded as: time needed to center the rover plus time to record and to change to the next station and then multiplied by the number of the stations. Finally, the time needed for the total station measurement was 98min and that of RTK measurement was 44min. When comparing required time of the two methods, total station was consumed more time than RTK.

Table 2: Adjusted Traverses of TS and RTK Data by Civil3D Software

Line		TS		RTK	
		Distance m	Azimuth DSM	Distance m	Azimuth DSM
6	1	164.76	98° 59′ 13.7″	164.75	99° 0′ 7.5″
2	3	172.87	14° 8′ 2.9″	172.76	14° 8′ 6.7″
1	2	109.82	27° 25′ 26″	109.76	27° 26′ 17.3″
5	6	128.49	198° 30′ 59.7″	128.37	198° 33′ 2.5″
4	5	209.70	201° 43′ 20.1″	209.61	201° 43′ 20.0″
3	4	157.46	299° 23′ 28.3″	157.30	299° 23′ 44.3″

Table 3: Angular Misclosure Error Of Traverse For Both Devices

Estimated Errors	RTK	TS
Angular error DSM	0-00-00	0-00-00
Error North m	0.049	-0.020
Error East m	-0.029	0.074
Absolute error m	0.057	0.077
Error Direction	N 30-25-33 W	S 74-46-33 E
Precision	1 in 16351.85	1 in 12171.93
Standard Deviation SD	2.53	3.19
Point Separation Distance m	0.057	0.077
Angular error most probable at	point 3	point 6
East RMS m	0.018	0.039
North RMS m	0.021	0.096

Table 4: The Observed and Adjusted Traverse Coordinates by Total Station and RKT Devices

Points	TS				Points	RKT			
	Measured E m	Measured N m	Adjusted E m	Adjusted N m		Measured E m	Measured N m	Adjusted E m	Adjusted N m
T2	410456.27	3542593.45	410456.23	3542593.56	R2	410456.22	3542593.50	410456.21	3542593.50
T3	410498.49	3542761.09	410498.46	3542761.20	R3	410498.41	3542761.03	410498.39	3542761.04
T4	410361.29	3542838.37	410361.27	3542838.48	R4	410361.35	3542838.24	410361.34	3542838.26
T5	410283.68	3542643.56	410283.68	3542643.66	R5	410283.77	3542643.51	410283.75	3542643.54
T6	410242.81	3542521.83	410242.88	3542521.82	R6	410242.93	3542521.81	410242.911	3542521.84
T7=T1	410405.64	3542496.08	410405.64	3542496.08	R7=R1	410405.64	3542496.08	410405.64	3542496.08

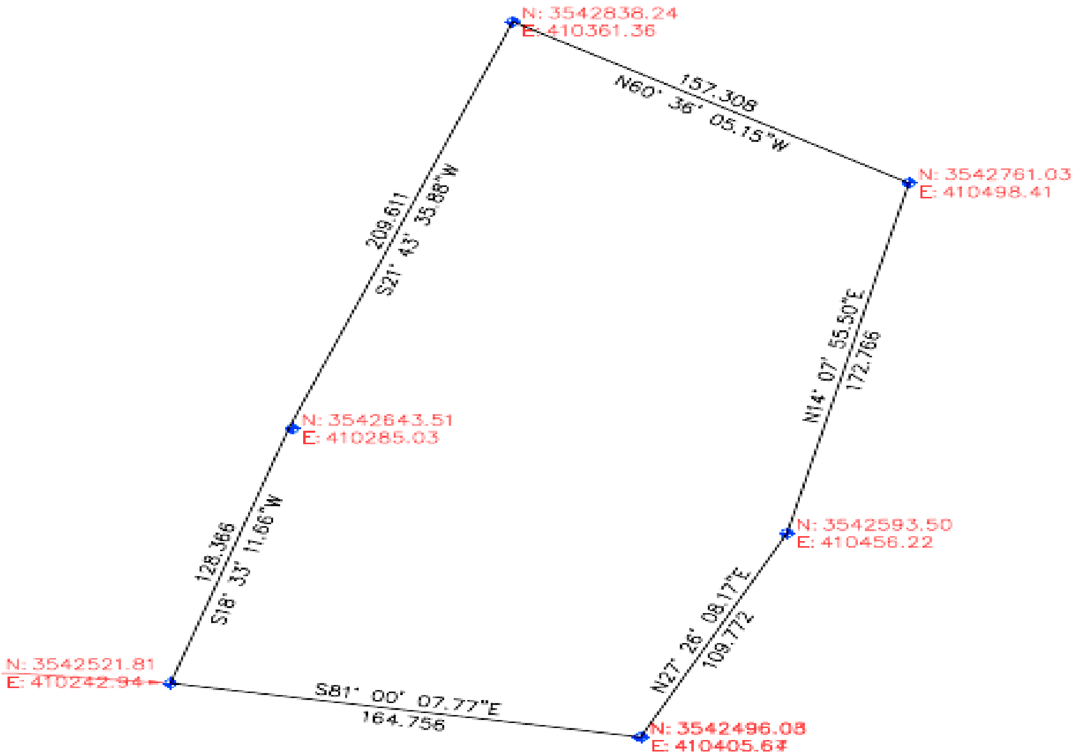

Figure. 5 The adjusted traverse drawn using Total Station observations.

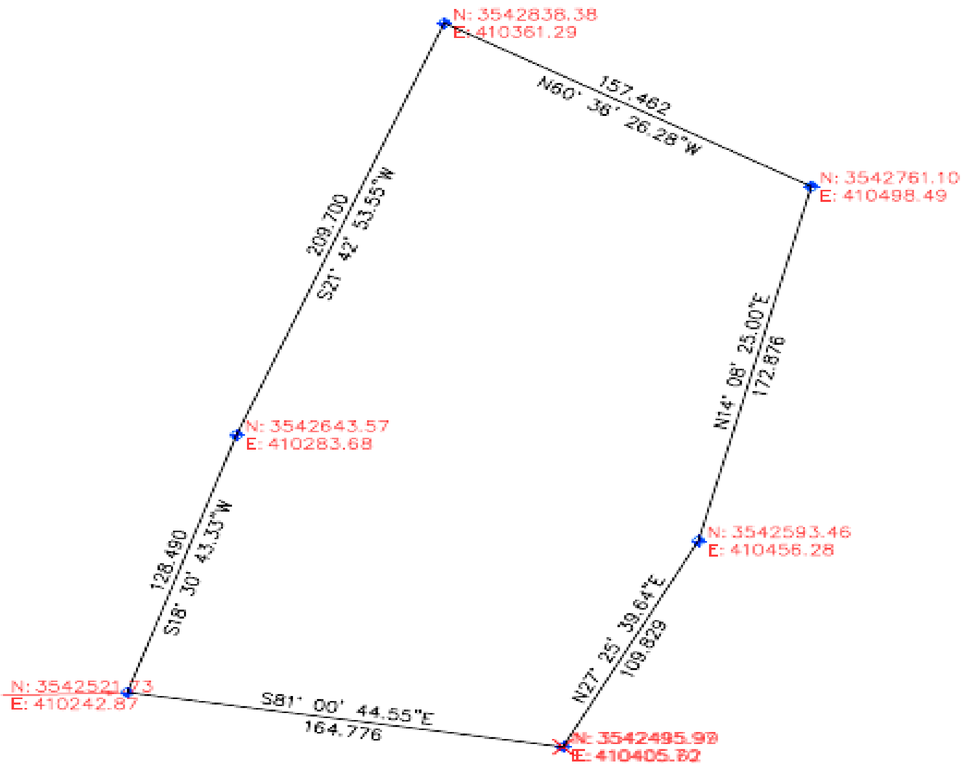

Figure. 6 The adjusted traverse drawn using RKT observations.

Table 5: Time expenditure for TS and RTK measurements .

Surveying Method	Man Power Required	Observation Time
RKT	1 man	98 minutes
TS	3 men or more	44 minutes

5. CONCLUSION

This study is focus on using different surveying instruments which are RKT GPS and TS to determine the accuracy assessment between them. The accuracy difference between the instruments is through adjustment of 6 points closed traverse. According to this study results the accuracy of the RTK device was better than the Total station device in both angular and linear misclosure errors with absolute errors 0.057m and RMSE for east 0.018m ,and north 0.021m besides, SD=2.53. RKT can measure points without any line of sight requirement, this makes RKT more effective tool for control point establishment. However RTK based survey is not only practical and efficient but also time saving than TS and more efficient in human resources. Therefore RTK is well suited to repetitive surveys and control surveys can be conducted considerably .

2. REFERENCES

[1]. Idris, M, ``Evaluation of GPS-RTKand Total Station for Topographic Survey and Strategic Decision in Private Companies" The 1st International Conference on Geodesy, Geomatics, and Land Administration KnE Engineering, PP, 323-332, 2019. DOI 10.18502/keg.v4i3.5874

[2]. Lin, L.S., "Application of GPS RTK and Total Station Systems on Dynamic Monitoring Land Use", Proceedings of the ISPRS Congress Istanbul, Turkey, pp,1-6, 2004.

[3]. El-Mowafy, A., " Performance Analysis of the RTK Technique in an Urban Environment, Australian Surveyor, Vol. 45, No.1, pp, 47-54, 2012.

[4]. Chekole, S. D." Surveying with GPS, Total Station and Terrestrial Laser Scanner: a Comparative Study", MSc thesis, KTH Royal Institute of Technology, 2014.

[5]. Elhassan, I. M., "Testing RTK GPS Horizontal Positioning Accuracy within an Urban Area", Journal of Engineering and Technology, RRJET , Vol. 6, No. 3,pp,17-24, 2017, ISSN:2319-9873.

[6]. Ameen, Q. M., andTais, A. S., "Evaluate Accuracy between RTK GPS and Total Station in Adjustment Closed Traverse ",International Journal of Interdisciplinary Environmental Studies. Vol.10, No. 10, pp, 1-15,2019.

[7]. Schofield, W.," Engineering Surveying", Sixth Edition, © W. Schofield 2007.

[8]. Chappell, E." AutoCAD Civil 3D 2014 Essentials" Autodesk Official Press. John Wiley & Sons, 2014.