Surveying with GNSS and Total Station: A Comparative Study

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Abstract: Today, advanced Global Navigation Satellite System (GNSS) receivers are improving the accuracy of positioning information, but in critical locations such as urban areas, places with big trees, high tension electric poles, marshes, high buildings, mountainous areas, and so on, the satellite availability is limited due to the signal blocking problem, which degrade the required accuracy. For this reason, different methods of measurement should be used.

The objective of this study is to evaluate and compare the precision, the accuracy, and the time expenditure of total station (TS), Global Positioning System (GPS), currently called (GNSS), (Static and Real Time Kinematic methods). Measurements will improve the knowledge about how much precision and accuracy can be achieved and at what time expense. To investigate this task, two Benchmarks (BMs) were established inside the campus of College of Technology Erbil polytechnic University (EPU) which are used as base lines for the establishment of a reference network consisted of 20 control points. Each point was measured five times by using Topcon 105 Total Station and served as a reference value for comparison with RTK-GNSS, GPS instrument. The data were processed from the instruments to a computer using different software according to the instruments used such as Topcon Tools v7, Trimble's GNSS Net, GIS Map Source Version 6.4, and Leica geo office software.

The registration and geo-referencing (using Arc Map 10.2.2. software) process was performed to convert the scanned satellite images of the point cloud to combine scanned maps together in one coordinate system then transformed to World Geodetic System 1984 (WGS84).

According to the obtained results, the reference network points measured with TS were determined with Easting13mm, Northing 11mm and Elevation 15mm precisions for both horizontal and vertical coordinates. When using RTK-GNSS method on the same reference network points, which is expressed by RMSE, the accuracy obtained for easting was 8 mm, for northing was 10.6mm, and 8.4 mm in elevation has been achieved. The RTK-GNSS measurements, which were measured five times, determined with a maximum standard deviation of Easting 0.9 mm, Northing 0.96 mm and Elevation 0.93 mm for horizontal and vertical coordinates, respectively. The precision of the remaining control points is below these levels.

The Center line of the road (120m) was projected (set out) using TS, then the points were measured with RTK-GNSS. The maximum difference between both methods in Easting was 19mm, in the Northing was 22 mm and in the Elevation was 30 mm. So, the differences considered to be small as it was in acceptable range.

Keywords: Global Navigation Satellite System (GNSS), Total Station (TS), Accuracy, Precision, GIS

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1. Introduction

1.1 Background of the Study

In recent years, with the development of technology, the TS, static GNSS, RTK-GNSS together with other surveying instruments such as Scan Station, aerial Photography are used for many tasks within different works, for example, geodesy, engineering, architectural and mining surveys, military, navigation, aviation, and documentation of cultural heritage together with different accuracy level depending on the needed requirements. (Sjöberg, 2012).

This study deals with evaluation and comparison of precision, accuracy, and time expenditure of two surveying instruments and methods followed for each instrument. These instruments are Total Station (TS), Global positioning system (GPS), and currently called Global Navigation Satellite System (GNSS). Later the surveyed points measured and corrected by the above two methods were placed on geo referenced satellite images, also the scanned images using Arc Map 10.2.2 software for the purpose of comparison. Accuracy and precision for those in the surveying profession as well as other technical and scientific fields are defined in different ways; accuracy refers to how closely a measurement or observation comes to measure a true or established value, since measurements and observations are always subject to errors. Precision refers to how closely repeated measurements or observations come to duplicate the measured or observed values.

Accuracy of surveying techniques using devices such as GNSS, TS, and other means of surveying are dependent on a number of parameters that limit their measurement quality. For instance: multipath, the inherent satellite signal accuracy, signal transmission delay, receiver hardware and software limitations, satellite signal obstruction are some of the problems associated with GNSS measurement. On the other hand, limitations stemming from TS are computed coordinates in local or target coordinate system, the reference surface for measuring height is geoids. Because of earth's curvature, the accuracy of TS measurement can also be affected by distance limit (the accuracy will decrease when increasing the distance).

1.2 Aim and Objectives of the Research

The aim of this study is to evaluate and compare the accuracy, precision, and cost (time expenditure) of two methods: GNSS and TS. The research attempts to achieve the following object:

1. Using Static GNSS, and TS for determination and fixing the BM and reference network
2. Determine and evaluate of the reference network which can be served as a reference value for comparison with RTK-GNSS, and TS3-
3. Determine and evaluate accuracy and precision of RTK-GNSS and TS methods.
4. Determination of the adjusted values of coordinates of points in a network surveyed by TS, using the method of least square by minimizing the sum of square of residuals.
5. Compare the results of the methods based on Standard Deviation SD and Root Mean Square Error RMSE analysis.

1.3 Significant of Surveying and Restriction Problems

The recent geodetic GNSS receivers are improving the accuracy of positioning information, but in critical locations such as urban areas or forests, the satellite availability is difficult due to the signal blocking problems, such as trees, high building, high tension electric poles, multipath etc., which degrade the required accuracy. As a result of this reduction of data, accuracy of the final result will be altered. TS can measure a single point coordinate precisely, but the computed coordinates are in local or target coordinate system, which needs datum transformation. When the required data is from the datum, the accuracy is affected by angle and distance of sight, weather condition, and other factors. Considering these limitations, the research will evaluate and compare accuracy, precision and time expenditure of these two surveying methods (TS and GPS-GNSS).

2. Overview

2.1 Total Station (TS)

Total Station (TS) is a precise surveying instrument used in the survey process that works for all types of surveying works such as location, setting out of road works, airports, railways, buildings, waterways, canals, sewerage, and other engineering projects.

The TS instrument combines all the aspects of advanced electronic theodolites with coaxial EDM. Horizontal and Vertical angles are obtained in the range of 1 to 10 second, depending on the model of the instrument. Distance accuracies are in the range of (5mm ± 5 to 10ppm) to (2mm ± 1ppm). Furthermore, TSs are equipped with microprocessors and has built-in programs and software to watch instrument status, control advanced functions, monitor the operation of the instrument, recalibrate as needed, and facilitate a wide variety of available software applications programs (Kavanagh,2011).

2.2 Receivers

There are different types of receivers as follows GNSS receivers' range in ability and cost from survey-level (millimeter) receivers capable of use in surveys requiring high accuracy and high cost.

Figure (1) shows the Leica GNSS SYS 300. System includes the following specification: SR 9400 GNSS single-frequency receiver, with AT 201 antenna, and CR333 controller, which provides software and collects data for real-time GNSS. It can provide the following accuracies: ± ((10–20) + 2 ppm x D) mm (Distance) using differential carrier techniques; and 0.30–0.50 m using differential code measurements (Courtesy of Leica Geo systems, Inc.) (Kavanagh, 2011).

Figure 1: GNSS EPOCH 50 Used in the Research

3. Study Area, Methodology and Data Collection

This section is related to explore the data that has been collected from the field by different methods and showing the results with comparing thereof methods.

3.1 Study Area

For The purpose of conducting detailed survey and data collection, the college of technology campus, and the area around the campus were specified as illustrate in Figure. (3-12). The area for data collection inside the campus and the area around the campus were about 48,000 and 600,000 square meters respectively.

A reconnaissance survey for the project area was fulfilled and followed by establishing two BMs and a reference network of 20 control points, which have been used as a reference value for the detail survey. Moreover, the network was established using Topcon ES 105 version TS. Figure. (2) shows the project area and the two BMs locations, also shows relocating the CP24 BM (established by CORS at Darato main Road), to BM1, and BM2 in the campus using static GPS and TS methods. Figure. (3) shows the ER01BM (Shanadar) which was also transformed to BM1 and BM2.

Figure 2: Study area boundary

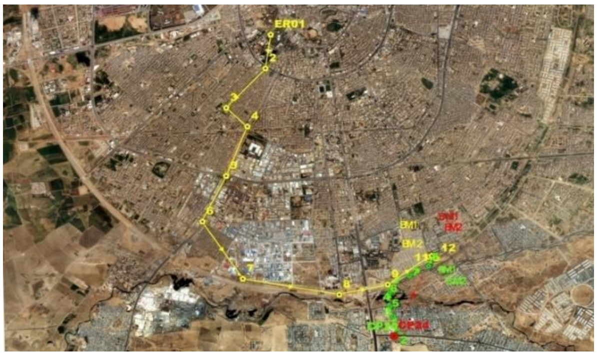

Figure 3: Study area location of the two BMsCP24 and ER01

3.2 Satellite Images Registration and Geo Referencing

In order to register the image, an old satellite image which is previously registered and geo-referenced was used; the coordinates have been transformed into UTM WGS 84 system through geo-referencing using Arc Map 10.2.2 software. A new registered and geo-referenced image was obtained, then it was used to identify the surveyed observation using the two methods TS, and GNSS.

3.3 Methodology for Establishing Benchmarks and Reference Network

In order to evaluate the accuracy and precision of the collected data, initially two BMs were relocated inside the University campus by using both GPS and TS, and the BMs reference is the two permanent fixed locations which were established previously by Iraq CORS (Yenter et al., 2005). The first BM locates in front of Shanader Park (ER01), and the other is at the midland of dual carriageway to Daratoo sub district (CP24). Then a network of control points which is used as a reference was established for comparison with different methods of measurement (GPS, and TS). The reference network consists of twenty control points, which are established by using a Leica 1201 TS. Each control point has been measured five times, to calculate high precision of network, the two new BMs, which were used to establish the reference network, were also measured with static GPS in order to transform the datum from the local coordinate system to the required coordinate system, WGS84 UTM Zone Iraq38N system, Thus, this network served as a reference value.

3.4 Assessment of Accuracy and Precision

The measurement for accuracy and precision are evaluated by computing the individual measurement of RMSE and SD (Catherine A. Peters, 2001). This can be computed from the deviations between the real and the measured values. True value of the measured quantity is determined with significantly higher precision. In this study the coordinates of the reference network, which are determined up to 1mm level, were considered as 'real'. RMSE was computed by using the following formula:

$$\text{RMSE} = \sqrt{\sum_{n=1}^{n} \frac{(\hat{\imath} - li)}{n}} \qquad [1]$$

Where:

$\hat{\imath}$ is the established value,

li is individual measurement and n is number of measurements.

The SD is a variation of the repeated measurement, i.e., the precision of each individual observation. This can be computed from the individual measurement and the mean values of the individual measurement; it is computed using the following formula:

$$\mathrm{SD(l)} = \sqrt{\sum_{i=1}^{n} \frac{(\hat{\imath} - \mathrm{li})^2}{n-1}}\ ,\ \hat{\imath} = \sum_{i=1}^{n} \frac{\mathrm{li}}{n} \qquad [2]$$

Where:

$\hat{\imath}$ is true or established value,

li is individual measurement, $\hat{\imath}$ is mean value of the measurements and n is number of measurements. (Chekole, S. D. 2014).

3.5 Location of the Two BM s from CP24 Using RTK-GNSS and TS

The observed and computed coordinates with their SD of the permanent point CP24 and the two BMs are presented in Table (1) and Table (2):

Table 1**:** Coordinates of CP24, BM1, BM2 and their SD transformed by Static GNSS

Point	Easting(m)	Northing(m)	Ellipsoid. Height(m)	SD Easting(m)	SD Northing(m)	SD Elevation(m)	Scale Factor
CP24	412518.705	3998526.509	427.156	0.0003	0.0005	0.0011	0.999692575
BM1	413241.547	4000162.340	435.493	0.0005	0.0006	0.0012	0.999692575
BM2	413319.686	4000108.018	435.363	0.0001	0.0002	0.0003	0.999692575

Table 2: Coordinates of CP24, BM1 and BM2 transformed by TS

POINT	Easting(m)	Northing(m)	Elevation(m)
CP24	412518.705	3998526.509	427.156
BM1	413241.559	4000162.369	435.482
BM2	413319.689	4000108.038	435.322

3.6 Location of the Two BM s from ER01 Using RTK-GNSS

Another fixed-point coordinates which are (ER01) located close to City Centre in Shanader Park was observed by RTK-GNSS method using Leica-15 GNSS instrument to transform the coordinates to the two BMs inside the Erbil Technical College as indicate in table (3), which shows the coordinates of BMs using RTK-GNSS.

Table 3: Coordinates of ER01, BM1 and BM2 transformed by RTK-GNSS

P GNSS	Easting(m)	Northing(m)	Elevation(m)
ER01	410143.324	4004665.213	415.857
BM1	413241.557	4000162.354	435.432
BM2	413319.698	4000108.038	435.321

The results of coordinates of BM1 and BM2 obtained by the two methods are shown in table (4). The Coordinates observed with static GNSS were used for all other observations made as they gave the most appropriate result when fixed on the permanent reference points CP24 and ER01.

Table 4: Coordinate of BM1, BM2. transformed using TS, RTK-GNSS, and static GNSS Adjustments and precision consideration

Point	Static GNSS(m)			TS (m)			RTK-GNSS(m)		
	E	N	Z	E	N	Z	E	N	Z
BM1	413241.547	4000162.340	435.493	413241.559	4000162.369	435.482	413241.557	4000162.354	435.432
BM2	413319.686	4000108.018	435.363	413319.689	4000108.038	435.322	413319.698	4000108.038	435.321

3.7 Network Observation Survey

After establishing the reference network and the targets, the network observation survey was started. GPS RTK measurements were taken on the two established BMs and the network in order to compare the results with TS measurement results. Also, measurements for handheld GPS and tape measurement for the points were taken and the results were compared. In order to evaluate the precision of the measurements, all network points and target points were measured five times. In addition to this, the time requirement was recorded during all the measurement for the purpose of comparison.

4. Calculation, Data Processing and Results

4.1 Calculation & Evaluation of Precision and Accuracy of TS Result

The measurements were started by setting up the TS on BM1 and entering the adjusted coordinates obtained previously from the two methods of transforming CP24 and ER01. Then, as it is described previously and directed to BM2 as resection, after those measurements were taken five times to compute the precision of all other points. The result is shown in Table (5), it can be seen that the level precision has been achieved for all coordinates. Results from TS measurements are tabulated in both easting, northing and elevation coordinates. Table (5) presents the mean values of five measurements and their standard deviations, together with the RMSE values to find the accuracy of the measurements using Equation (1), and standard deviation formula Equation (2) mentioned above.

As the result shows in Table (5), the maximum standard deviations are less than 13mm in easting coordinate, 11 mm in Northing coordinate, and they reach 15 mm in elevation coordinate, which indicates that the repeated measurements were quite close to each other.

To evaluate how much TS measurements were close to the established value, RMSE of the TS measurements were also computed as indicated in Table (5). This RMSE indicates the accuracy of the TS measurements of the reference network. Accuracy of the Easting coordinates ranges between maximum 11.6 mm and minimum 8.9 mm, Northing coordinates ranges between 9.9 mm and min 0.0 mm, and the accuracy of the elevation coordinates ranges between maximum 1.34 mm and minimum 0.0 mm.

Table 5: TS measurement, RMSE and SD

Point	TS Mean Value (m)			RMSE(m)			SD (m)		
	Easting	Northing	elevation	Easting	Northing	elevation	Easting	Northing	elevation
BM1	413241.559	4000162.369	435.482	0.00040	0.00048	0.00096	0.0005	0.0006	0.0012
BM2	413319.689	4000108.038	435.322	0.00008	0.00016	0.00024	0.0001	0.0002	0.0003
1	413008.851	4000897.325	434.203	0.00270	0.00141	0.00000	0.00302	0.00158	0.00000
2	412997.217	4000916.697	434.054	0.01169	0.00536	0.00000	0.01307	0.00599	0.00000
3	412997.715	4000920.698	433.988	0.00471	0.00531	0.00055	0.00527	0.00594	0.00061
4	412986.476	4000909.761	434.065	0.00207	0.00851	0.00000	0.00232	0.00952	0.00000
5	412984.420	4000913.963	434.028	0.00084	0.00228	0.00000	0.00094	0.00255	0.00000

14	13	12	11	10	9	8	7	6
413402.854	413389.396	413388.413	413399.068	413398.832	413399.991	413396.032	413404.190	413407.353
4000267.145	4000277.169	4000276.569	4000254.288	4000248.938	4000239.863	4000245.932	4000240.330	4000237.993
436.064	435.915	436.100	436.036	436.023	436.041	436.021	436.060	435.937
0.00466	0.00311	0.00726	0.00644	0.00339	0.00672	0.00339	0.00219	0.00358
0.00740	0.00396	0.00993	0.00730	0.00000	0.00212	0.00000	0.00089	0.00313
0.00000	0.00045	0.00055	0.00000	0.00000	0.00000	0.00000	0.00000	0.00134
0.00521	0.00348	0.00812	0.00720	0.00379	0.00752	0.00339	0.00245	0.00400
0.00827	0.00443	0.01111	0.00816	0.00000	0.00237	0.00000	0.00100	0.00350
0.00000	0.00050	0.00061	0.00000	0.00000	0.00000	0.00000	0.00000	0.00150

15	413510.049	4000309.870	437.016	0.00089	0.00321	0.00000	0.00100	0.00359	0.00000
16	413585.873	4000362.012	438.586	0.00942	0.00627	0.00055	0.01054	0.00701	0.00061
17	413588.540	4000363.911	438.678	0.00332	0.00152	0.00084	0.00371	0.00170	0.00094
18	413588.838	4000368.497	438.817	0.00351	0.00192	0.00000	0.00392	0.00215	0.00000
19	413598.289	4000368.394	438.815	0.00764	0.00351	0.00045	0.00854	0.00392	0.00050
20	413601.418	4000368.526	438.868	0.00371	0.00195	0.00055	0.00415	0.00218	0.00061

4.2 Calculation and Evaluation of Precision and Accuracy of RTK-GNSS

On the reference network, RTK-GNSS measurements were taken in order to compare them with the TS measurements. Using RTK method, all control points were surveyed five times so as to evaluate the precision of the measurements. To compute the precision of the repeated measurements of the reference network, RMSE of the RTK measurements were also computed using Equation (1), and for standard deviation Equation (2) in order to evaluate how much the measurements were close to the established value.

As the result shows in Table (6), the standard deviations are less than 10 mm in Easting coordinate, 9.64 mm in northing, and they reach 9.39 mm in Elevation coordinate, which indicates that the repeated measurements were quite close to each other. The result of other studies done by Jonson et al (2003), and Chekole (2014) indicated that the SD for the horizontal coordinate was 9mm, 8mm respectively and 15mm and 20mm for vertical coordinates respectively. So, by comparing the results of this study with above mentioned studies results, the precisions of the horizontal and vertical coordinate are in mm and cm level respectively which are quite close to each other.

The calculated standard deviations result of RTK-GNSS measurements are quite close to the calculated standard deviation of the TS measurements.

To evaluate how much RTK-GNSS measurements were close to the established value, shown in Table (6), the RMSE indicates the accuracy of the RTK-GNSS measurements of the reference network. The accuracy of the Easting coordinates ranges between maximum 8.05 mm and minimum 1.1 mm, Northing coordinates range between maximum 10.62 mm and minimum 0.71 mm, and accuracy of the elevation coordinates ranges between maximum 8.4 mm and minimum 1.67 mm. This result was compared with the works of (Ehsani et al, 2004) and (Chekole 2014), in which, a horizontal accuracy of 10mm and 9mm are obtained, respectively. By comparing the accuracy of horizontal coordinates, it becomes clear that they are close to each other. It may be said that the results of this study are quite reasonable considering the errors attributed from satellite blocking, centering errors and so on.

Table 6: RTK GNSS measurement, its RMS, and standard deviation

Point	RTK-GNSS(m)			RMSE(m)			SD		
	Easting	Northing	elevation	Easting	Northing	elevation	Easting	Northing	elevation
BM1	413241.557	4000162.354	435.432	0.00040	0.00048	0.00096	0.0005	0.0006	0.0012
1	413008.841	4000897.336	434.216	0.00351	0.00462	0.00497	0.00392	0.00516	0.00556
2	412997.213	4000916.708	434.049	0.00365	0.00313	0.00336	0.00408	0.0035	0.00376
3	412997.728	4000920.703	433.98	0.00286	0.00217	0.00316	0.0032	0.00242	0.00354
4	412986.47	4000909.767	434.051	0.00444	0.00212	0.0046	0.00496	0.00237	0.00515

5	412984.435	4000913.975	434.043	0.00321	0.00071	0.00467	0.00359	0.00079	0.00522
6	413407.341	4000237.995	435.924	0.00610	0.00711	0.00594	0.00682	0.00795	0.00664
7	413404.179	4000240.338	436.05	0.00268	0.00336	0.00167	0.003	0.00376	0.00187
8	413396.022	4000245.929	436.014	0.00336	0.00261	0.00383	0.00376	0.00292	0.00429
9	413399.982	4000239.861	436.032	0.00477	0.00217	0.00439	0.00534	0.00242	0.00491
10	413398.842	4000248.935	436.038	0.00585	0.00212	0.00526	0.00654	0.00237	0.00588
11	413399.063	4000254.282	436.033	0.00462	0.00502	0.00313	0.00516	0.00561	0.0035
12	413388.416	4000276.574	436.099	0.00182	0.00529	0.00212	0.00203	0.00592	0.00237
13	413389.407	4000277.159	435.919	0.00805	0.00241	0.00303	0.009	0.00269	0.00339

14	413402.844	4000267.143	436.058	0.00288	0.00261	0.00219	0.00322	0.00292	0.00245
15	413510.035	4000309.858	437.029	0.00586	0.00862	0.00628	0.00655	0.00964	0.00703
16	413585.879	4000362.024	438.573	0.00638	0.0041	0.0084	0.00713	0.00458	0.00939
17	413588.549	4000363.921	438.682	0.00110	0.00663	0.00661	0.00122	0.00742	0.00739
18	413588.83	4000368.495	438.825	0.00522	0.00541	0.00483	0.00584	0.00605	0.0054
19	413598.3	4000368.389	438.821	0.0076	0.01063	0.00415	0.00849	0.01188	0.00464
20	413601.421	4000368.52	438.855	0.00295	0.00698	0.00709	0.0033	0.0078	0.00793

5. Conclusions and Recommendations

5.1 Conclusions

Based on the data observed and data processing analyzing the following conclusion are drawn

1. Based on the results obtained, accuracy of the reference network with TS determined was less than 1.3 mm standard deviation both for horizontal and vertical coordinates for all points. This result has been achieved because of the round measurements and two face measurements of the TS. On the same control points of the network, RTK-GNSS method was performed and according to the result obtained, the standard deviations for Easting was 9 mm in northing

11.8mm and 9 mm in elevation coordinate, this indicates that the repeated measurements were quite close to each other. The accuracy of the RTK measurements on the network, which is expressed by RMSE are, easting 8 mm, in northing 10.6 mm, and 8.4 mm in elevation coordinates, meanwhile the accuracy obtained using TS observations was represented by RMSE, are 11.69 mm for northing, 9.93mmin easting and 1.34 mm in elevation coordinates.

2. For location of the centerline of the 120m road opposite to Erbil Technical College campus, some differences were found. The maximum difference in Easting was (19 mm), Northing was (22 mm) and in elevation was (27 mm), so, the differences considered to be small as it was within the acceptable range.
3. The time expenditure summarized as more time (196 minute) was consumed for TS measurement and in the Static-GNSS method, but almost similar time (167 minute for RTK-GNSS) was consumed.
4. In order to evaluate the quality of the measurement, absolute value of each coordinate difference between each method should not exceed ($k*\sigma d1$), which limits the errors not to be beyond certain limit by multiplying their sigma differences with constant k 2.776 using T-test statistical evaluation. Based on this quality control measure, more than 95% of the total result has achieved the requirements. This can be interpreted as values which lied within the allowable limit (interval limit), considered as accepted values. But values out of the interval limit considered as risk values, which might contain gross errors. There was no point found out of the interval limit, so all the points were accepted.
5. When using RTK-GNSS measurement, small tripod was used to erect the rover vertically. Initially, it was expected to achieve accuracy in mm level. But, due to some errors like centering error, instrumental error, receiver satellite signal, obstruction, by trees and high building or high-tension electric pole, some results have been deviated into cm level.

5.2 Recommendations

It is expected that the obtained results from this study will improve the knowledge about accuracy, precision, and time consumption of the three methods used (TS, RTK-GNSS and static GNSS). Therefore, it can be decided which instrument should be used for which specific application depending on the presented results. For further improvement of accuracy, the following recommendations are forwarded:

1. TS should be calibrated at some regular intervals since there were problems in the level bubbles; one on the tribrach and the other on the TS that could not be leveled at the same time. So, once the instrument is calibrated, it will improve the level of accuracy.
2. Better accuracy and precision can be achieved by calibrating all those instruments before the measurement campaign.
3. In applications which require high precision, so as to serve as reference value, such as control point establishments; this study recommends using TS instead of RTK-GNSS, if the number of points are too much, otherwise using static GNSS for small number of points or when fixing BMs.
4. It was very difficult to manage the field measurement alone, specially establishing the reference network which faced big problems. There are possibilities of occurring gross errors and therefore, working in group is recommended.

5. When collecting data using RTK-GNSS in places close to the high-tension electric poles, high building, high trees, or marsh land, it is recommended to use RTK-GNSS in combination with TS, as there will be obstruction and interference of waves, so the reception of signals is weak.
6. In case of locating of roads and fixing the centre lines with slope stakes it is recommended to use the combination of both the TS and RTK-GNSS. This combination will reduce the errors and gives more precise results and avoid the obstacles.

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