Height Evaluation and Linear Accuracy of Digital Level, Total station, GPS and Orthophoto

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ABSTRACT

In this research, Digital level (DL), Total station (TS) and GPS were used to assess accuracy and precision of the height component. Field observations were implemented in two tested areas. A reference network which consisted of 34 points on area1 and 10 control points on area2 which had been observed five times using Digital level, RTK-GPS and Total station (TS) where Digital level was considered as a base for comparison. Several known control points were used as check points to evaluate the accuracy of measurements. According to the obtained results, TS and GPS-RTK measurements were compared with the adjusted reference points measured by precise Digital level (DNA 03). Around ±15 mm standard deviation for TS and ±13.5 mm for GPS were achieved. Linear accuracy of TS, GPS, and orthophotos measurements from Vossing German Company were also investigated in regular features within the same tested areas. The actual lengths were measured with steel tape up to a millimeter accuracy and were considered as being a base for comparing. The maximum deviation 22mm accuracy has been obtained in area2 and 12 mm in area1. The study shows that the extracted features from orthophotos had less accuracy in hilly regions due to relief displacement whereas they were more accurate in gentle slopes.

KEYWORDS : Digital level, Total station and DGPS, Orthophoto, Linear accuracy, Analysis and Comparison.

1. INTRODUCTION

Surveying has been an important factor in the development of the human environment for many centuries. It is a prerequisite requirement in the planning and implementation of nearly every form of construction. The significant development of surveying methods have added flexibility in data and information collection and enabled potentially a wider range of applications. Moreover, surveying techniques played a significant role in the progress of 3D coordinates of the point of interest whereby increasing its application. In many applications, the role of surveying has been intended to be used for better accuracy. The term accuracy is common in several applications to express the quality of observations, measurements and computations. Sjöberg (2012), explained that the

required accuracy depends on the need of achieving production which were divided into three categories such as low, medium and high accuracies. Low accuracy includes common navigation works on the sea, position and velocity in small scale geophysical exploration. Medium accuracy contains navigation in coastal waters, earthquake survey and hydrographic survey. The applications that require high accuracy are engineering construction projects, precise continuous height control, deformation monitoring and precise hydrographic surveying and etc. The accuracy of surveying methods used in this research, GPS, TS, Digital level, and aerial photos depend on a number of elements that limit the quality of measured data. For example: GPS positioning errors include satellite position, satellite clock, and receiver clock, signal delay due to troposphere and ionosphere, and multipath error as well as the limitations of receiver hardware and software (Amin, 2004). Regarding the limitations from the total station (TS), the accuracy of measuring data is affected by distance limit, the shorter the distance the better the accuracy that can be achieved. Whereas the accuracy of digital level depends on the orthogonally of the barcode rods over the point of interest and the distance from the rod to the instrument. Finally,

accuracy of aerial photos depend on the field of view, altitude and etc. i.e. images taken from nadir (vertical photos) is more accurate than oblique images. The accuracy of survey instruments have been investigated and assessed by many researchers. In the work of Lin (2004) the accuracy of GPS-RTK and total stations were tested. The results indicated that the positional accuracy of GPS-RTK was better than the TS, about 1.4 cm achieved when the GPS-RTK was used and the possibility of 1.6 cm was determined when TS was used. Ehsani et al. (2004), tested a 0.5 square kilometer area with RTK-GPS, the base station and four reference points that had been established in the highest point of the selected area. Corrections for GPS signals were received in real time from a base receiver at a known position to all rover receivers' points. The results of their work show that a horizontal position accuracy of 1 cm was achieved by compensating for atmospheric delay (Ionosphere and troposphere), and other variable errors in GPS geometry. In contrast to GPS, TS can give better results in urban areas, due to the obstruction of satellite visibility such as urban canyon, tree canopy and tall buildings. These obstructions degrade the signal of GPS in those urban areas. In such cases TS are used, this was clearly shown in Borgelt S C. et al. (1996). They reported that the accuracy of TS compared with RTK-GPS in open sky can give standard deviation about of 0.12 m in vertical position. While in obstructed areas, better accuracy was obtained using TS, which was below 0.5cm. Two areas, with good and poor GPS visibility were selected for assessing the quality of RTK. The result showed that the quality of RTK-GPS with good GPS visibility were better than obstructed areas (Abdulrahman, 2013). Accuracy of different RTK-GPS instruments such as Leica, Topcon and Trimble, were tested on nine control points by Jonsson K.O. et al. (2003), total station was also applied on the same points. The results achieved from GPS-RTK measurements in a horizontal and vertical components were (0.1 and 2) cm respectively. The work conducted by Ahmed (2012), GPS-RTK with TS measurements were tested on a network implemented in the campus of Royal Institute of Technology (KTH), School of Architecture and the Built Environment, Stockholm, Sweden. The objective was to assess the RTK-GPS achievable accuracy, which tested the standard deviation of the observations under different satellites configurations and evaluated RTK-GPS achievements in urban areas. The test showed that the assessment of accuracy and precision of the RTK-GPS was accomplished by comparing the coordinates of the observed points with that of separately accurate ones using TS. The result indicated that the difference between the coordinates of the total station and RTK-GPS were found to be (2 and 3) cm for the horizontal and vertical coordinates respectively. While the results conducted by Chekole. (2014) on the same tested area were 1.8 cm for both horizontal and vertical coordinates. To sum up the aforementioned works, the limitation of GPS, TS and Digital level also have different time consumption in the field works. Therefore, the aim of this paper is to assess and compare precision and accuracy of the above survey techniques. The objective of this study is to focus on collecting data from field measurements using different surveying techniques: Digital level, Total station, RTK-GPS and extract linear measurements from orthophotos.

2. THE SURVEY INSTRUMENTS

The Global Positioning System (GPS) used was Leica viva GNSS receiver,

Figure (**1**, a, which measures the incoming phase of the satellite signals to high levels of precision in repeated measurements (Kostov, 2011). The Leica TS02, total station was used,

Figure (**1**, b. The total station is a digital theodolite integrated with an electronic distance measurement (EDM) to read slope distances from the instrument to a particular point and reduce it to horizontal by observing the vertical angle. The Leica DNA03 digital level was also used,

Figure (**1**, c, leveling with digital levels and barcode rods is the technique used to establish vertical control points. The accurate results were obtained when all systematic errors were controlled. Short sight lengths and balanced sights were the most limiting restrictions.

Figure (1) : a) Leica GPS receiver GS10 Base station and GS15 rover, b) Total station and c) Digital level DNA03

3. WORK PLAN

1. Two areas were tested within the University of Duhok campus, area 1 was approximately flat and in the second area there were large variations in slopes. The natural point features were recognized well on orthophotos. The aim was to evaluate the accuracy and precision of the instrument survey used in this study to provide accurate horizontal and vertical position in such areas.

2. Several well identified points were distributed in two selected areas and their corresponding points appeared on orthophotos. For this reason an accurate reference height was created based on the first order vertical control point where the orthometric height was known with the aid of precise leveling procedure using Leica Digital level DNA03. The leveling procedure started from the first order point and closed to the control point (FP05). This point was established by Vossing German Company in Duhok region as illustrated in Figure (2. The height of all selected points in the two tested areas were compared with the GPS and total station.

3. Several known control points were used as check points for evaluating the quality of measurements.

4. Extracted linear features from orthophotos of the two selected areas were used for evaluating the accuracy of orthophotos compared to the GPS and TS.

Figure (2) : Left. First order vertical control point, right. FP05, Vossing, German control point in Duhok

4. TEST AREA AND DATA COLLECTION

The study area is located in two test sites around the university campus, university of Duhok. First, reconnaissance of the two selected areas of study were performed, and followed by fixing 34 and 10 control points as reference points for both selected areas respectively, which have been used as a reference value for a detailed survey. These points have been established using Leica TS02, GPS, and Digital level DNA03 for measuring the height of all points of the selected areas.

Figure (3 shows the study area and the reference control points. The control point, C12 and C2 were previously surveyed by the university's survey team, 2011 using GPS in static mode with high accuracy and it was considered as a true GCPs.
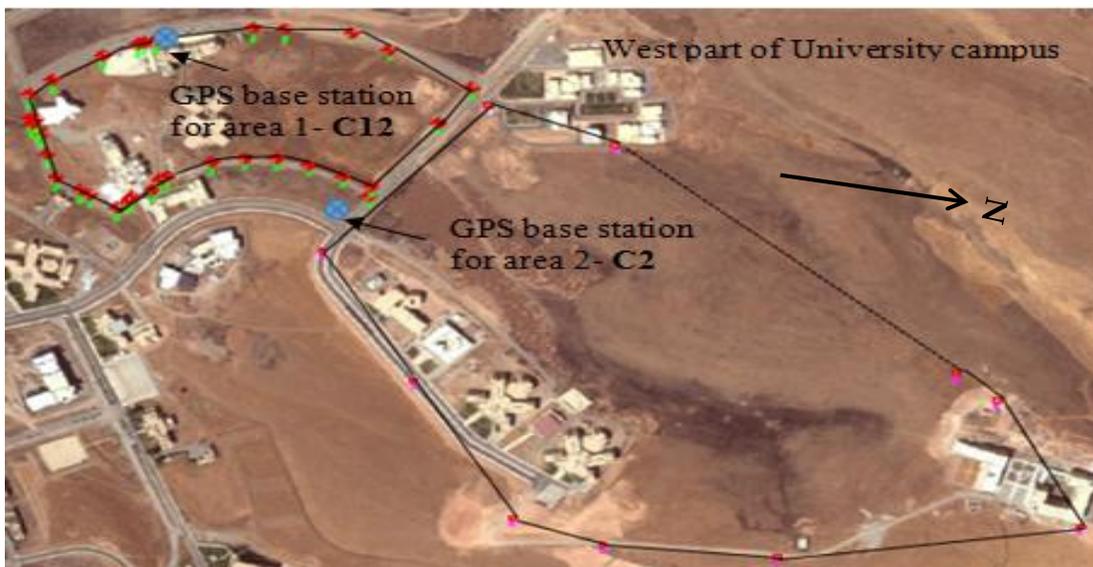
Figure (3) : The location of the control points in the two selected areas

A. Field Measurements of the Two test Areas

Points in the first test site were distributed over an area of 300 x 540 m and height range between 486 m and 504 m. the selected points were well defined featured points e.g. road curb corner, building corners and road intersections. The survey loop works were carried out from control point C12 and closed to the starting point (C12). The points of the second area spread over 450x700 m and the height range was between 504 m and 557 m. The survey works start from the study area1 passing through the roads around the building which linked to the first study area as illustrated in

Figure **(3**. All of these points were measured on the GPS system, Total Station and precise Digital level. The Leica viva differential GPS was used in two test areas as mentioned before. In area 1, the Leica GS10, base station was set up on a highly accurate control point, C12, and its known coordinates can be shown in Figure (**4**, left. This control point (C12) was used as a base station to provide and establish reference points in the area of interest to be used for comparison using Leica GS15 in RTK mode as shown in Figure (**4**, right.

Figure (4) : left, Control pointsC0, C1, C2 and C12 with its known coordinate. Right: GPS-RTK set up on the Reference point C2 and C12.

The GNSS, GS15 receiver was used as a rover for measuring and recording data on the selected points for comparison with total station. The vertical component of these points was also measured. The coordinate system of the point was in WGS84. The spheroid height of all recorded points were converted to geodetic height by subtracting the average difference =17.62 m which is the separation between spheroid and Geoid in the selected area of study. This value is not constant but varies depending on the local geoid in Iraq. This technique (GPS- RTK) was implemented to measure and record all the required points in two selected areas using the known coordinate points (C12) and (C2). The system measured the selected points five times to obtain the reported positional accuracy which was around 8 mm. The height of all points in the study area were based on the first order control point where the orthometric height was known (**486.8159 m**). The levelling procedure started from the first order point ID (45-16) and closed to FP-05 control point, Figure **(2**. This control point was used for validating the quality of geodetic height in the study area. Differential leveling was carried out on selected points on the two

tested areas using Precise Digital level DNA03 with the precision of 0.2mm. A closed-loop leveling procedure was adopted. The measurement was repeated three times, and the average value was taken and stored automatically by the digital level device. The elevation errors obtained from the leveling process were within the allowable accuracy. Since the permissible closure for a level circuit is based upon the lengths of lines or the number of setups, it is logical to adjust the elevation on this basis to get the required precision.

B. Results

Table 1 shows the deviation in height value between GPS and reference (GPS $_{elev.}$ – DL $_{elev.}$) Points in area1 have a maximum deviation in the range of -4cm to -8 cm and increase to 18 cm in point F28. While, the maximum deviation in height obtained via TS (TS $_{elev.}$- DL $_{elev.}$) in the range of ± 1cm to ± 4cm and increase to 22cm in point F28. The RMSE of both instruments GPS and TS are close around ± 0.038m. When the point F28 remove from the measurement, the RMSE improved to ±0.015 for TS and ±0.013 for GPS in height measurements.

Table (1) : the deviation in height value between GPS and reference value (GPS $_{elev.}$ – DL $_{elev.}$)

Point	Digital Level elevation (DL elev.)	GPS Elevation	ΔGPS, DL	TS elevation	ΔTS, DL
C1	515.5305	515.526	-0.0045	515.5762	0.04565
F1	515.3244	515.3184	-0.006	515.3484	0.024
F2	522.6382	522.6301	-0.0081	522.6501	0.0119
F3	522.5609	522.5543	-0.0066	522.5343	-0.0266
F4	518.5549	518.5372	-0.0177	518.5672	0.0123
F5	515.8727	515.8581	-0.0146	515.8881	0.0154
F6 (c2)	517.867	517.8681	0.0011	517.8981	0.0311
F7	513.8917	513.8799	-0.0118	513.919	0.0273
F8	512.5346	512.523	-0.0116	512.563	0.0284
F9	511.9193	511.896	-0.0233	511.936	0.0167
F10	511.363	511.346	-0.017	511.376	0.013
F11	509.7168	509.6933	-0.0235	509.7333	0.0165
F12	509.3316	509.3024	-0.0292	509.3424	0.0108
F13	508.5591	508.5233	-0.0358	508.5633	0.0042
F14	507.1896	507.1553	-0.0343	507.1953	0.0057
F15	506.7915	506.7595	-0.032	506.7895	-0.002
F16	505.8153	505.7806	-0.0347	505.8206	0.0053
F17	504.3583	504.3127	-0.0456	504.3527	-0.0056
F18	503.9598	503.9175	-0.0423	503.9475	-0.0123
F19	502.9875	502.9463	-0.0412	502.9763	-0.0112
F20	502.1044	502.0659	-0.0385	502.0959	-0.0085
F21	501.585	501.5477	-0.0373	501.5777	-0.0073
F22	501.238	501.2048	-0.0332	501.2448	0.0068
F23	501.1673	501.1398	-0.0275	501.1698	0.0025
F24	501.1473	501.0993	-0.048	501.1293	-0.018
F25	501.2249	501.1893	-0.0356	501.2193	-0.0056
F26	503.371	503.3375	-0.0335	503.3675	-0.0035
F27	504.7158	504.6833	-0.0325	504.7233	0.0075
F28	505.9011	506.0835	0.1824	506.1235	0.2224

F29	506.1254	506.0825	-0.0429	506.1225	-0.0029
F30	507.3777	507.3415	-0.0362	507.3815	0.0038
F31	507.6004	507.554	-0.0464	507.594	-0.0064
F32(c12)	508.0989	508.072	-0.0269	508.112	0.0131
F33	512.6362	512.6145	-0.0217	512.6545	0.0183
F34	514.0425	514.0097	-0.0328	514.0397	-0.0028
		RMSE	**0.0379**		**0.0395**

The height deviation of GPS and TS in Area2 compared to reference height (Digital level) as shown in Figure (**5** has less accuracy than area 1. This can be explained by large variations in slope and the wide length between selected points.
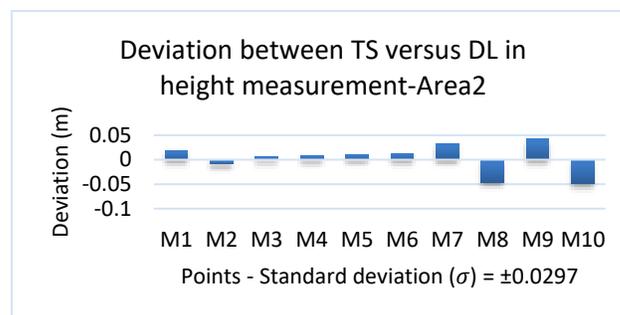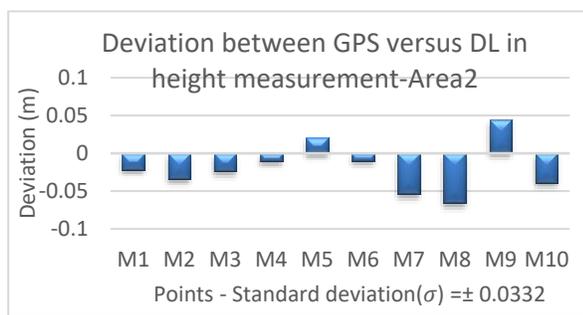
Figure (5) : Deviation in height measurements, university site area2. Left: GPS versus Reference height (DL). Right: TS versus Reference height.

5. THE QUALITY OF GPS IN THE AREA OF STUDY

In the previous section, 4 a, the GPS- RTK technique was implemented to compare the accuracy of all points in two selected areas. RTK was used to measure the selected points five times to get better 3D quality, reported by the receiver, which less than 8 mm. This 3D quality refers to the accuracy of the GPS measurement in relation to the base station. The value of the 3D quality can be small or large depending on the satellite availability and other sources of errors that affect the GPS measurement. The Equation 1 below express the 3D quality (σ_{enh}).

$$\sigma_{enh} = \sqrt{\sigma_e^2 + \sigma_n^2 + \sigma_h^2} \tag{1}$$

Where:
σ is the standard deviation of E, N and h coordinates

RTK measurements were taken to compare with the total station measurements; all selected points were surveyed five times so as to evaluate the precision of the measurements. In order to evaluate how much the measurements were close to the established value, the standard deviation and RMS of the RTK measurements were computed using Equation 1.

The results showed that the standard deviations were less than 8 mm in the horizontal coordinate and they reached 16 mm in the vertical coordinate as shown in figure 6, left, which indicated that the repeated measurements were close to each other. These results were better than the results that were obtained by Jonsson K.O. et al. (2003), the standard deviations for the horizontal and vertical coordinate were 9 mm and 20 mm respectively. While the results of RMS errors of RTK measurements, which indicated the accuracy of the RTK measurements of the reference network (see figure 6, right), showed that the accuracy of the horizontal coordinates were about a maximum of 12 mm (points F15,20,21,25, 27 and 34) and minimum of 5 mm (point F22). The point F14 was the only point that reached 15 mm. The accuracy of the vertical coordinates ranged between a maximum of 23 mm (point F29) and a minimum of 12 mm (point F11). These errors were attributed to the obstruction of satellite visibility to the GPS receiver. By comparing the results of the accuracy of the horizontal coordinates with the work conducted by Ehsani et al. (2004), their results were about 10 mm achieved, which were close to each other. This closeness of the results can be explained by compensating for atmospheric delay, orbital errors and other variables in GPS geometry.
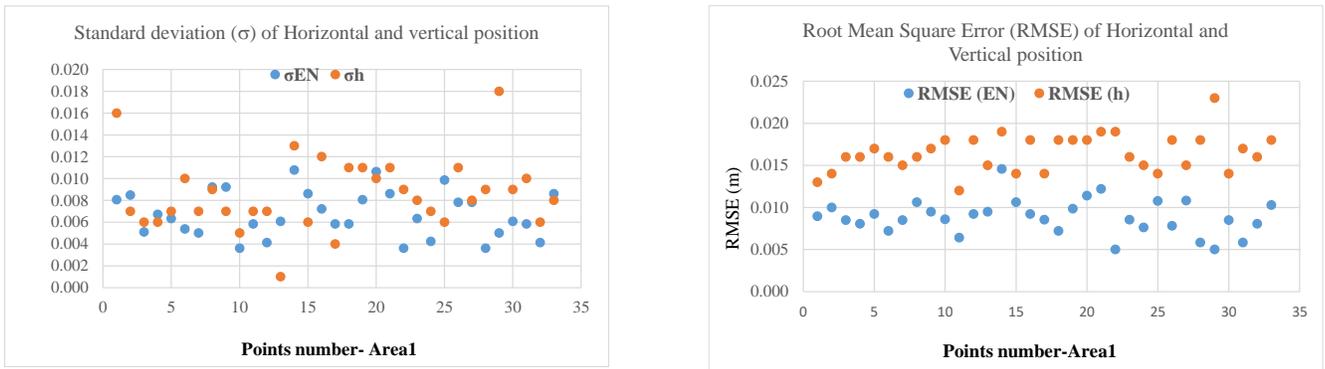
Figure (6) : GPS- RTK measurements: Left. Standard deviations. Right. RMSE -Area1

The accuracy of RMSE and standard deviation of each measurement in area 2 is listed in Table (**2** and the results were better than area1. This is attributed to the visibility of GPS signals to satellites which were better than area 1 (see figure 3).

Table (2) : GPS-RTK measurement, its RMSE and standard deviations -Area2.

Point	Easting	Northing	Height	E	N	H	σE	σN	σH
	Mean value of RTK			**RMSE**			**Standard deviation (σ)**		
M1	314160.044	4081318.212	523.308	**0.007**	**0.008**	**0.017**	**0.007**	**0.006**	**0.008**
M2	314226.572	4081485.767	534.150	**0.007**	**0.008**	**0.012**	**0.005**	**0.008**	**0.015**
M3	314406.865	4081101.408	521.907	**0.005**	**0.007**	**0.013**	**0.005**	**0.005**	**0.003**
M4	314628.448	4081221.141	530.138	**0.005**	**0.006**	**0.012**	**0.003**	**0.003**	**0.006**
M5	314862.189	4081350.542	550.638	**0.008**	**0.007**	**0.013**	**0.007**	**0.002**	**0.004**
M6	314909.087	4081468.695	562.926	**0.008**	**0.007**	**0.022**	**0.002**	**0.003**	**0.001**
M7	314930.097	4081697.052	583.653	**0.003**	**0.006**	**0.015**	**0.001**	**0.001**	**0.002**
M8	314877.199	4082093.428	606.289	**0.007**	**0.008**	**0.017**	**0.007**	**0.006**	**0.008**
M9	314663.408	4081985.125	594.615	**0.003**	**0.004**	**0.12**	**0.001**	**0.003**	**0.006**
M10	314616.142	4081931.075	587.641	**0.004**	**0.006**	**0.015**	**0.003**	**0.001**	**0.004**
				±0.002	**±0.00**	**±0.03**	**±0.002**	**±0.002**	**±0.004**
				RMSE (enh) = ±3mm			**σ(enh)=± 4.9 mm**		

A. Measurements of TS and GPS versus Check points

The RMSE and standard deviation for the position (ENh) were also calculated between the RTK mode and Total station coordinates versus check points in the two study areas and the results are shown in Table 3 using the Equation 2.

$$RMSE\ (enh) = \sqrt{\frac{\sum_{i=1}^{5}(A-B)^2}{n}}\ ;\quad RMSE\ (enh) = \sqrt{\frac{\sum_{i=1}^{5}(A-C)^2}{n}}$$
(2)

Table (3) : RMSE and standard deviation of the position for each measurement versus check points

Test area	Check points position enh (m) (A)	Position enh (m)	
		enh (m), TS02 (B)	enh (m), RTK-GPS (C)
Area1: $RMSE\ (enh) = \sqrt{\frac{\sum_{i=1}^{5}(A-B)^2}{n}}\ ;\quad RMSE\ (enh) =$			
RMSE		0.124	0.435
σenh		0.222	0.345

Area2: $RMSE\ (enh) = \sqrt{\frac{\sum_{i=1}^{5}(A-B)^2}{n}}$;	$RMSE\ (enh) =$	
RMS	0.233	0.322
σenh	0.244	0.127

B. Accuracy Analysis

To verify the quality of measurements, the mean value of 10 measurements of the TS and RTK measurements were compared and the results showed that the compatibility between the two measurements were accurate and precise. Table (**4** shows that the maximum deviation in the horizontal and vertical position of both areas between TS and RTK were 5 mm and 11mm respectively.

Table (4) : The difference between TS and RTK mean in two selected areas

Point/ area1	TS mean- RTK mean				Point/ area2	TS mean-RTKmean			
	ΔE	**ΔN**	**ΔEN**	**ΔH**		**ΔE**	**ΔN**	**ΔEN**	**ΔH**
F2	-0.002	-0.003	0.004	0.006	M1	-0.003	0.004	0.005	0.011
F3	0.001	-0.004	0.004	0.007	M2	0.004	-0.003	0.001	0.010
F5	-0.003	0.002	0.004	0.005	M3	-0.001	-0.001	0.002	-0.008
F7	-0.001	0.002	0.002	-0.008	M4	0.001	0.002	0.001	0.006
F10	0.004	-0.003	0.005	0.011	M5	-0.001	-0.001	0.003	0.005
F12	-0.002	0.004	0.004	0.007	M6	-0.002	0.002	0.003	0.004
F17	0.003	-0.001	0.003	0.009	M7	0.001	0.003	0.005	0.003
F22	0.004	0.003	0.005	0.006	M8	-0.003	-0.004	0.004	-0.011
F24	-0.002	-0.003	0.004	0.004	M9	0.002	-0.003	0.001	0.009
F26	-0.004	-0.003	0.005	0.008	M10	-0.001	0.001	0.005	0.005

The standard deviation of the difference between total station and RTK of the ten measurements were also calculated using Equation 3 and 4. The results showed that the maximum difference in standard deviation of horizontal position was ± 9 mm and ±17 mm in vertical position as shown in

Figure (**7**.

$$d_{(GPS,RTK-TS)i} = Corrd._{TSi} - Coord._{GPS,RTKi} \qquad (3)$$

$$\sigma_{d(GPS,RTK-TS)i} = \sqrt{\sigma_{TSi}^2 + \sigma_{GPS,RTKi}^2}$$

(4)

Where;$d_{(GPS,RTK-TS)i}$ is the difference between TS and GPS, RTK coordinates, $\sigma_{d(GPS,RTK-TS)i}$ is the standard deviation of $d_{(GPS,RTK-TS)i}$.
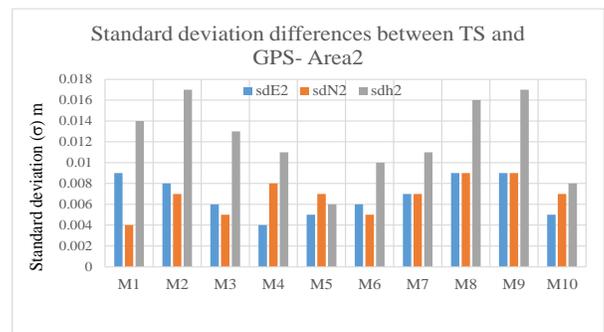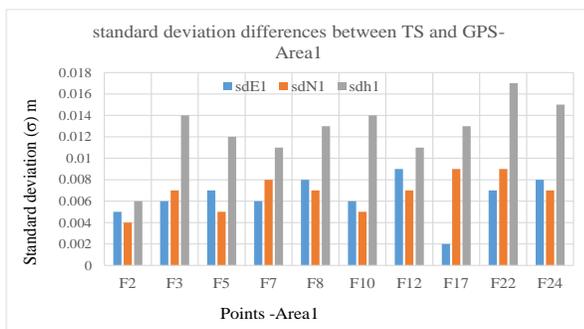
Figure (7) : Standard deviation differences between TS and GPS in two selected areas. Left: Area 1. Right: Area2

Figure (**7** shows that the maximum standard deviation of the differences between total station and RTK in the two selected areas for horizontal and vertical components, where in area1 11mm in point (F12) and 17 mm in point (F22) were detected respectively. In area 2, the points detected were 12 mm in point (M8 and M9) and 17mm in points (M2 and M9). The maximum 3D deviation between TS position and RTK position in both areas were 22 mm. While the minimum 3D deviation of the differences between TS position and RTK position in both areas were around 7mm in area1 and 8mm in area2. The average standard deviation (σh) differences in vertical position and the total of 3D deviation

(Tσenh) in the two selected areas were close to each other. This means that there were no significant differences in measurements as shown in Figure (**8**.
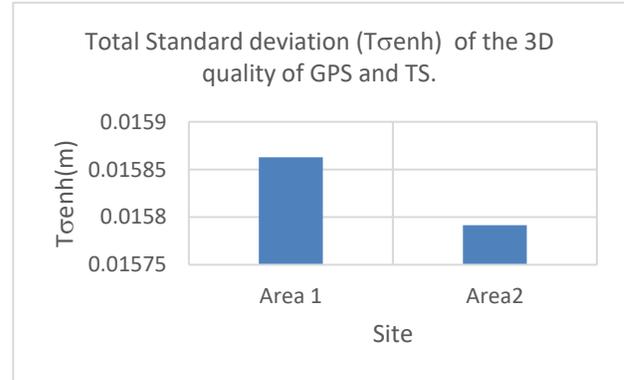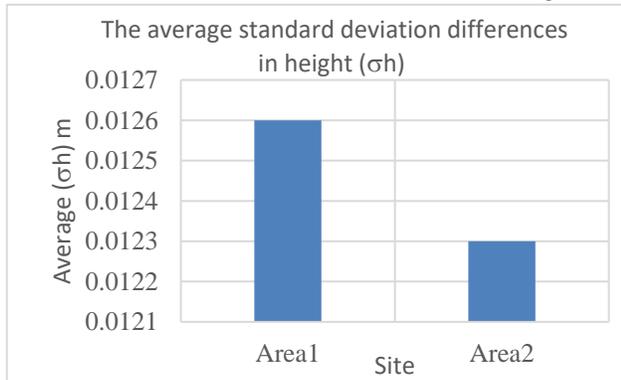
Figure (8) : Left: The average differences of standard deviation of vertical position, and Right: The total standard deviation of the differences in position between GPS and TS in two area of study.

6. LINEAR **ACCURACY**

Linear accuracy was assessed between surveys instruments used in this paper relative to the actual length. The actual length was measured with steel tape up to millimeter accuracy and considered to be a base for comparison. A test was carried out in regular buildings for the two selected areas of study as illustrated in Figure (**9**. The results were compared between measured lengths in relation to the base (steel tape) as can be shown in Table 5. The distances of each line as shown in Figure (**9** were computed using coordinates of each point measured by GPS and TS. While the length of those lines were measured directly using AutoCAD civil 3D software.

Figure (9) : Measured length of regular building highlighted in Orthophoto of the Vossing German Company. Left, area 1. Right, area 2

Table (5) : Linear accuracy of sensor recorded length-Area1

Regular Building Area1	Steel tape side length(m), (A)	Side lengths (m)			Differences (m)		
		TS02 (B)	DGPS (C)	Orthophoto (D)	A-B	A-C	A-D
1	21.500	21.480	22.250	21.44441	0.02	-0.75	0.05559
2	4.900	4.880	5.240	5.10905	0.02	-0.34	-0.2090
3	22.210	22.220	22.580	22.31728	-0.01	-0.37	-0.1073
4	9.870	9.820	9.880	9.81736	0.05	-0.01	0.05264
5	7.530	7.66	7.640	7.75665	-0.13	-0.11	-0.2266
6	42.070	41.950	41.080	42.23595	0.12	0.99	-0.1659
7	21.400	21.409	21.370	21.43246	-0.009	0.03	-0.0324
8	19.610	19.640	19.630	19.66377	-0.03	-0.02	-0.0538

9	15.500	15.804	15.520	**14.40705**	-0.304	-0.02	1.09295
10	13.850	13.841	13.820	13.90942	0.009	0.03	-0.0594
11	24.250	24.245	24.270	24.21164	0.005	-0.02	0.03836
12	15.000	15.060	15.120	15.15949	-0.06	-0.12	-0.1595
13	15.200	15.180	15.450	15.16462	0.02	-0.25	0.03538
14	41.120	41.114	41.100	41.13	0.006	0.02	-0.01
15	13.065	13.075	13.076	13.078	-0.01	-0.011	-0.013
Standard deviation (σ)					**0.095**	**0.361**	**0.312**
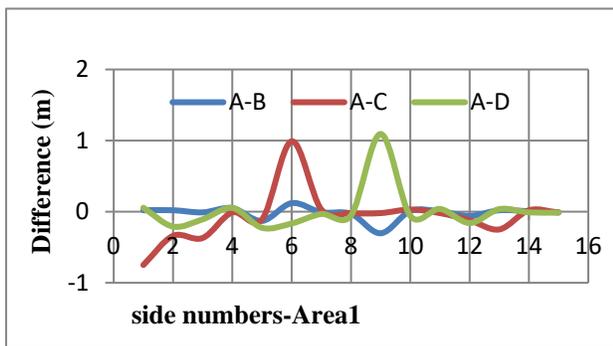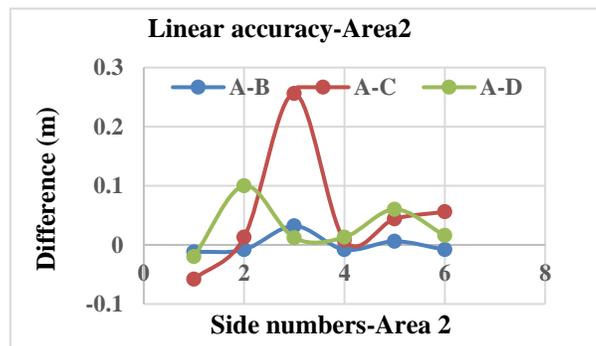
Figure (10) : the deviation in length between sensors relative to the actual length: left. Area1. Right.Area2

As shown in Figure (**10**, the high discrepancy in GPS and orthophoto for measured length relative to fixed length (length measured by steel tape) was clearly 1 m in area 1 which was a higher value relative to those measured in area2. The discrepancy in TS for measuring length in both areas relative to the fixed length was more accurate than GPS and orthophotos. This is because the GPS is not accurate when close to the building due to the obstruction of the GPS signal in satellite visibility. While the errors of the aerial photos' resulted from relief displacement in height variations of the selected area of study.

7.CONCLUSIONS

The purpose of this project work was to evaluate and compare accuracy and precision of surveying techniques (Digital level, TS, GPS and data from orthophotos). The comparison was carried out between Digital level versus TS and GPS-RTK on the reference network; and the accuracy of othophotos encountered to TS and GPS in measuring length on regular features such as buildings and other well defined features. To accomplish the objectives of the paper, three main tasks were implemented. Firstly: a network of 34 and 10 control points were established in two areas respectively with high precision (0.2mm) with Digital level according to the instrument specification, and served as a reference value. Secondly: on the same network, total station and RTK methods were performed to compare the result with that of the digital level. Finally, the linear accuracy of orthophotos of

Vossing German company were evaluated in comparison with the survey instrument (GPS and TS) used in this study. It is possible to collect the 3D data of the object rapidly and in detail by the survey instruments mentioned in this research. However, the important steps is to decide the type of sensor to be applied. If there are close range photogrammetry, for instance Terrestrial Laser scanner or a none-metric camera is used instead of orthophotos, they will produce better results. As seen in this study, the measurement differences between the two selected areas are inversely proportional to the accuracy of the sensors used and directly proportional to instrument distance to the objects.

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